PROGRESS REPORT (FIRST DRAFT)
U.S. Army Corps of Engineers Waterways Experiment Station
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STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

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Prepared for US Army Engineer, Waterways Experiment Station
Vicksburg, Mississippi 39180-6199

NOTE TO REVIEWERS

This is a first draft of a progress report describing the vertical strip drain test section in the Craney Island Dredged Material Management Area. It is estimated that the dredged fill and marine clay will reach 90 percent consolidation in February or March, 1994. As a result, the data pertaining to the test section will be augmented as consolidation progresses. The main purpose of submitting this progress report for review is to obtain suggestions for improving the data presentation and analysis prior to completion of the project. It is anticipated that the input of reviewers at an early stage will improve the final report. Upon completion of the test section and incorporation of the final test results, the report will be submitted for a final review. Thank you in advance for contributing to the improvement of this research.

EXECUTIVE SUMMARY

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of 49 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the dredged material. Consolidation of the dredged fill and foundation clay will also cause a large increase in soil shear strength. ~~The strength gain will allow perimeter dikes to be constructed to a higher elevation without setbacks or stability berms.~~ It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future. Preliminary results show that the dredged fill and foundation clay are undergoing substantial (0.9 to 1.2 m in 6 months) consolidation settlement. In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of dredged material management areas.

A supplemental ~~investigation~~ study by the PI will investigate the use of strip drains beneath exterior perimeter levees to improve existing stability conditions.

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.76456	cubic meters
feet	0.3048	meters
miles	1.60935	kilometers

$$9.81 \frac{kN}{m^3} = 62.4 \frac{lbs}{ft^3}$$

$$\frac{kN}{m^3} \times 6.36 = lbs/ft^3$$

$$\frac{lbs}{ft^3} \times 0.157 = \frac{kN}{m^3}$$

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PREFACE

A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of 49 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the dredged material. ~~Consolidation of the dredged fill and foundation clay will also cause a large increase in soil shear strength. The strength gain will allow perimeter dikes to be constructed to a higher elevation without setbacks or stability berms.~~ It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future. Preliminary results show that the dredged fill and foundation clay are undergoing substantial (0.9 to 1.2 m in 6 months) consolidation settlement. In summary, prefabricated strip drains appear to be an economical technique for increasing the storage capacity of dredged material management areas.

This research was conducted for the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi and the US Army District, Norfolk (NAO), Norfolk, Virginia during the period August 1992 to August 1993. The research was performed under Contract No. DACW 39-92-M-6666 between WES and Dr. Timothy D. Stark, an Assistant Professor of Civil Engineering at the University of Illinois at Urbana-Champaign. Dr. Stark supervised the research and wrote this report. Mr. Thomas A. Williamson, a Graduate Research Assistant at the University of Illinois at Urbana-Champaign, performed the analysis and data reduction.

General supervision in the GeoTechnical Laboratory (GL), WES, was provided by Dr. Jack Fowler, Soil Mechanics Division (SMD), Mr. W. Milton Myers, Chief, Engineering Group, SMD, Dr, D.C. Banks, Chief, SMD, and Dr. William F. Marcuson, III, Chief, GL.

General NAO supervision of the study was carried out by Mr. Sam McGee, under the guidance of Mr. Ronn G. Vann, P.E., Chief, Dredging Management Branch, Mr. Thomas C. Friberg, Chief, Operations Section, and Mr. James N. Thomasson, P.E., Chief, Engineering Division. Technical information was provided by Mr. Mathew T. Byrne, P.E., Geo~~T~~technical Engr Sec. ~~Branch~~ (~~GB~~ GS), NAO and Mr. David A. Pezza, P.E., Chief, ~~GB~~ GS, NAO.

The commander and Director of WES is Col. Larry B. Fulton. Technical Director of WES is Dr. Robert W. Whalin.

This report should be cited as follows:

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STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

PART I: CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

Background

1. The Craney Island Dredged Material Management Area (CIDMMA) is a man-made 10 km^2 site with a storage area of approximately 8.9 km^2 (Figure 1). Planned in the early 1940's, construction of Craney Island began in August 1954 and was completed in January 1957. Craney Island is the long-term management area for material dredged from the channels and ports in the Hampton Roads area. The CIDMMA is located near Norfolk, Virginia, in Portsmouth, Virginia.

2. Dredged material has been placed in the management area almost continuously since it was completed in 1957. The original design was for an initial capacity of about 76.4 million m^3 at an annual dredging rate of 3.1 to 5.4 million m^3. Based on an annual dredging rate of 3.8 million m^3, Craney Island was designed for a service life of approximately 20-years (1957 to 1977). Continued dredging in the Norfolk channel has required the capacity of Craney Island to be increased through three major dike raising efforts. The initial dike raising from El. +2.4 to El. +5.2 m Corps of Engrs Mean Low water (CEMLW) ~~MLW~~ occurred around 1974 with the second increase to El. +7.9 m around 1980. It should be noted that the water depth at the time of construction was approximately 3.1 m. The US Army Engineer District, Norfolk (NAO) is currently raising the perimeter dike system based on recommendations presented by Fowler et al. (1987). The west dike is being raised to El. +10.4 m CEMLW but this raising required the placement of a 305-meter-wide underwater stability berm along the outer toe of the dike (Figure 2) to ensure stability. ~~The top of the berm corresponds to the Mean Low Water level.~~ The perimeter dike in the northwest corner is being raised to El. +10.4 m CEMLW using a dike setback of approximately 137.3 m (Figure 3). The north and east perimeter dikes are being raised to El. +12.2 m CEMLW with setbacks from the dike perimeter road of 128.1 m and 137.3 m, respectively (Figures 4 and 5). Dike setbacks have resulted in approximately 0.1 km^2 to 0.2 km^2 of lost storage capacity during each dike raising. Figure 1 shows the location of these dike cross sections. After the third raising is completed, the perimeter dikes will be at their maximum height due to foundation stability.

3. Using plans developed by Palermo et al. (1981), interior dikes were built within Craney Island to create three containment areas (Figure 1) that would improve sedimentation in the compartment being filled and allow the other two compartments to desiccate and consolidate at a faster rate. Desiccation will be accelerated by the removal

Corps of Engrs Mean Low Water is 0.6 m below NGVD 1929, 1972 Adjustment, and 0.2 m below MLW (NOS).

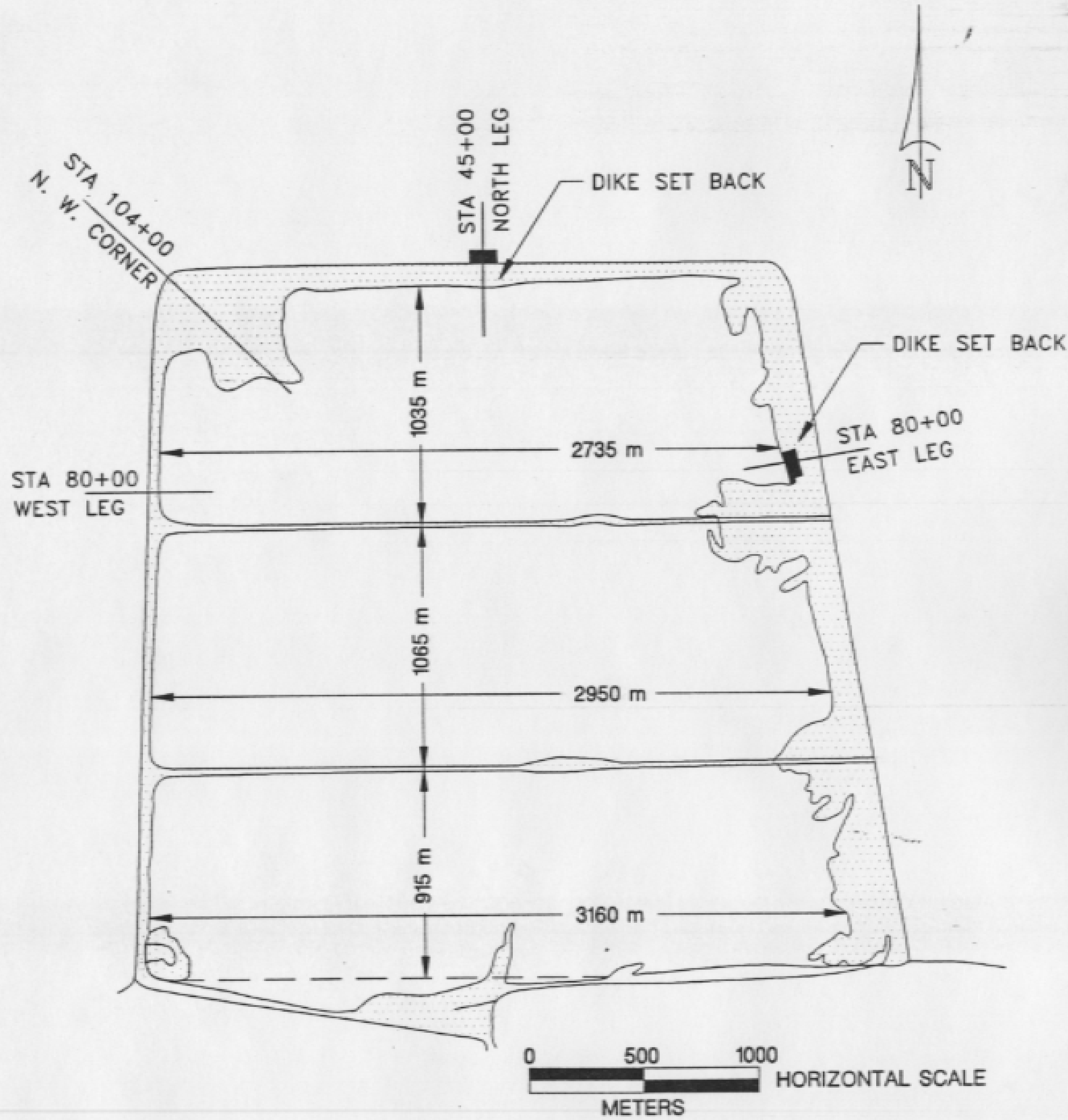


Figure 1. Plan View of Craney Island and Location of Dike Cross Section

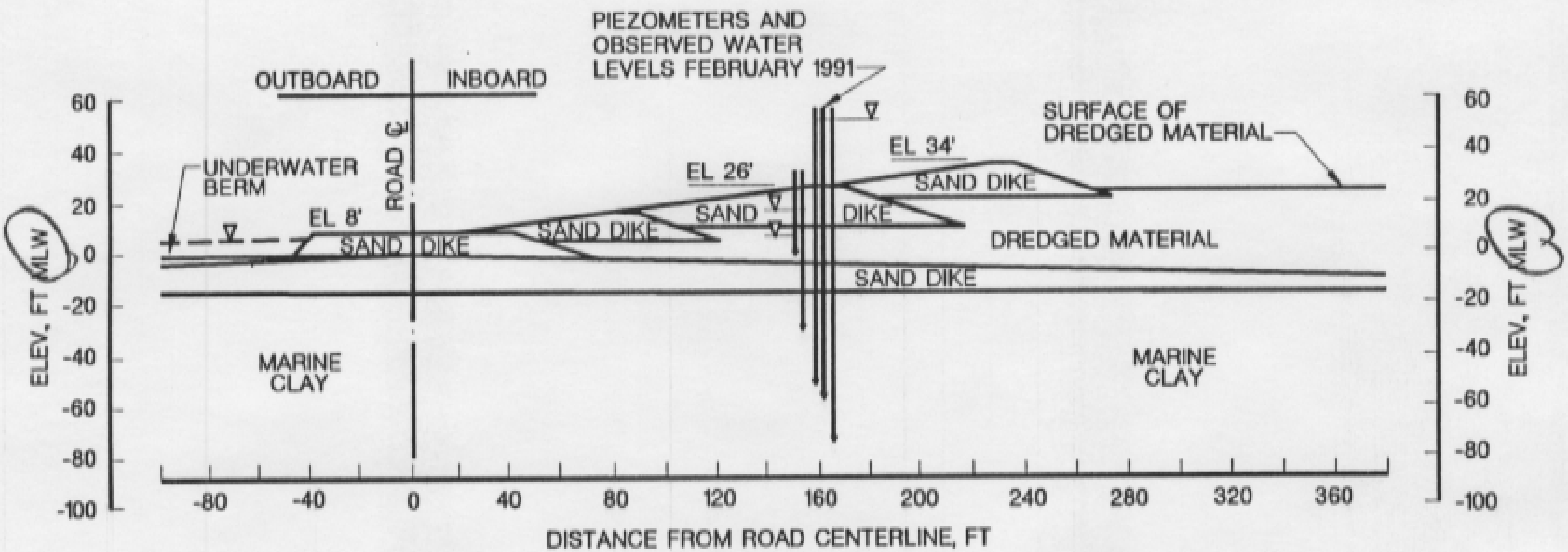


Figure 2. Generalized Cross-Section, West Perimeter Dike

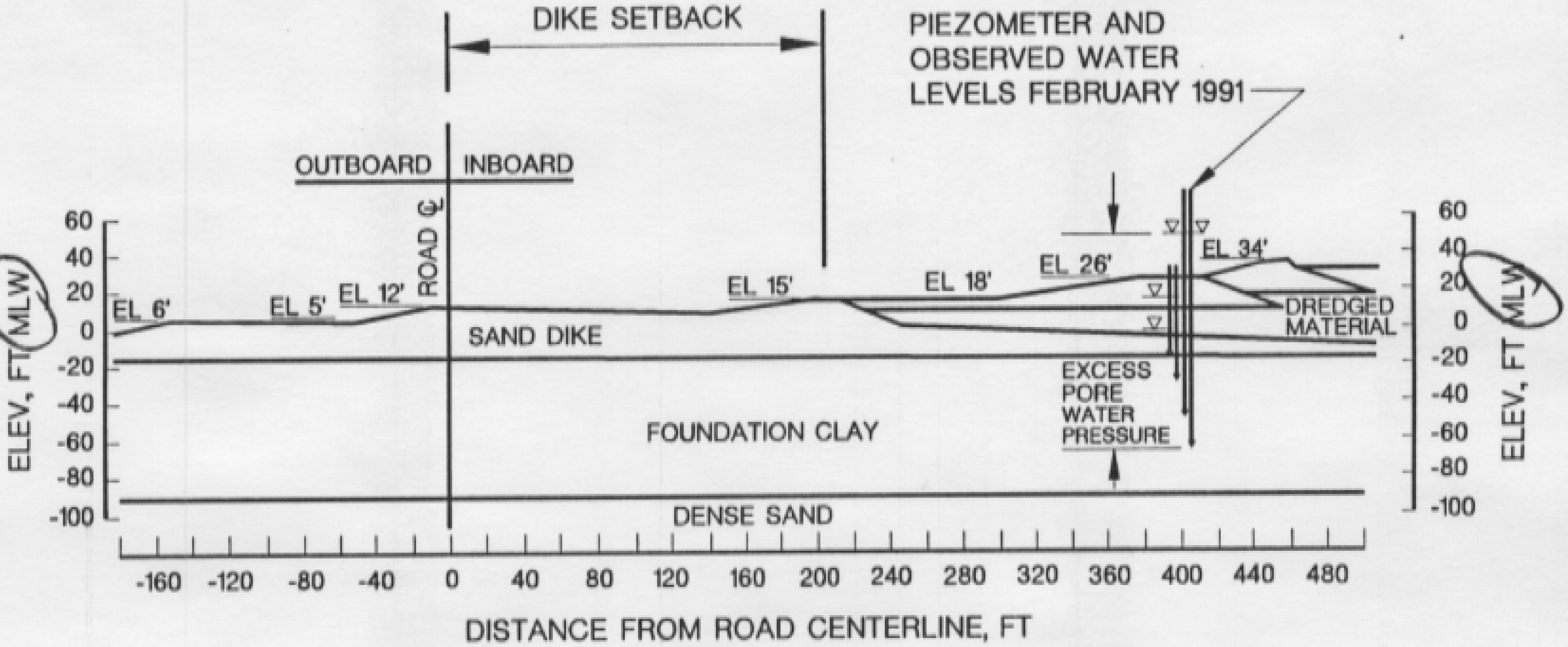


Figure 3. Generalized Cross-Section, Northwest Corner Perimeter Dike

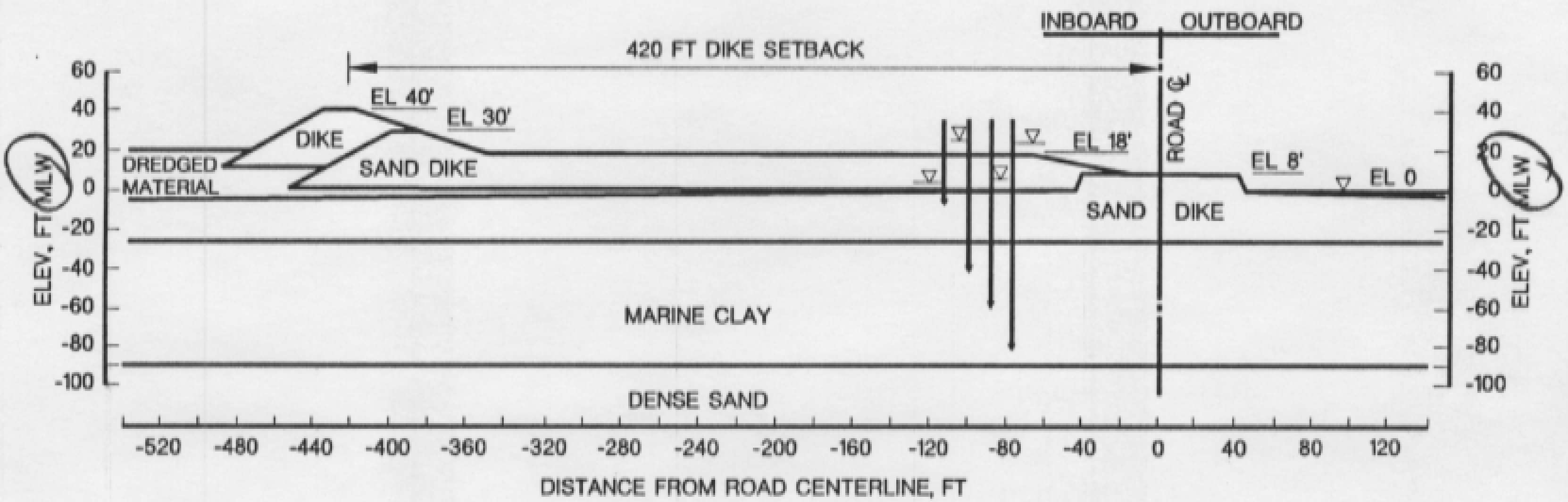


Figure 4. Generalized Cross-Section, North Perimeter Dike

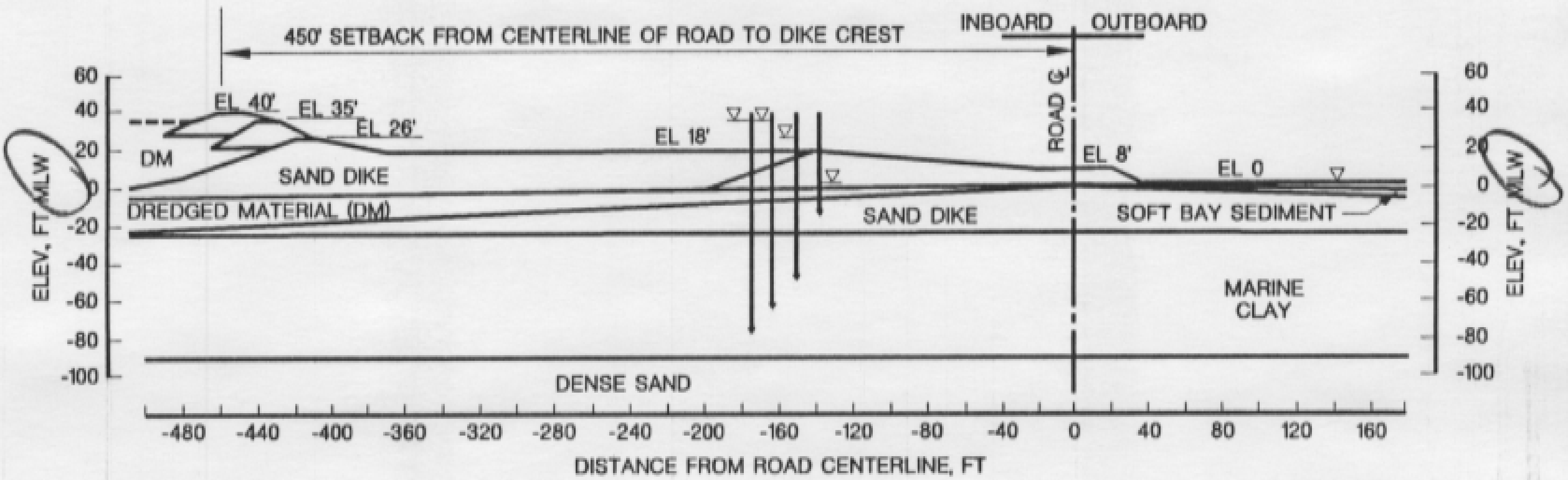


Figure 5. Generalized Cross-Section, East Perimeter Dike

and/or evaporation of surface water, and will increase the amount of consolidation because the effective density of the soil increases as the pore-water evaporates. Construction of the interior dikes was completed in 1983, and the dredged material management plan (Palermo et al. 1981) was implemented in 1984 starting with the center compartment. The management plan has resulted in each compartment being filled approximately every third year. On the average 3.1 to 3.8 million cubic meters of dredged fill is placed in a compartment each year. This results in an annual increase in dredged fill thickness of 0.9 m to 1.8 m in the compartment being filled. If the site was not subdivided, the annual increase in dredged fill thickness would be approximately 0.3 m to 0.6 m (Szelest 1991).

4. The Environmental Laboratory at the US Army Engineer Waterways Experiment Station conducted an extensive consolidation and desiccation analysis to predict the service life of the CIDMMA (Palermo and Schaefer 1990). This study utilized the finite strain consolidation microcomputer program PCDDF (Stark 1991; Stark and O'Meara 1991) and revealed that the current capacity of Craney Island will be exhausted around the year 2000. As a result, NAO began investigating new techniques for increasing the storage capacity of the CIDMMA.

Alternatives for Increasing Storage Capacity at Craney Island

5. Studies by Fowler et al. (1987) showed that the perimeter dikes are at their maximum height due to the current undrained shear strength of the marine clay foundation (Figure 6). However, if the undrained shear strength of the dredged fill and underlying marine clay was increased through consolidation, the perimeter dikes could be raised again. In addition, the increase in shear strength would probably allow the dikes to be raised without setbacks or stabilizing berms. The time required for this consolidation and strength gain is substantial, and thus would not alleviate the short-term storage problem.

6. An extensive study was conducted by Spigolon and Fowler (1987) on the feasibility of expanding the CIDMMA. Six expansion configurations were considered but the 1991 the Virginia State Legislature ruled that Craney Island could not be expanded or replaced at the present time. Therefore, the feasibility of restricting the usage of the CIDMMA to placement of unsuitable dredged material and ocean placing the suitable material was investigated (Stark 1993). The cost of ocean placement is approximately 9.95 $13.0 per m^3 whereas placement in the CIDMMA is about 0.92 $1.20 per m^3 (Szelest 1991). NAO is dredging at a rate of approximately 3.8 million m^3 per yr. Therefore, the difference between placement in the CIDMMA and ocean placement is approximately $44.8 million per yr. In addition, the environmental impact of ocean placing this large

$yd^3 \times 0.765 = m^3$

No longer used

Elevations are CEMLW not MSL as provided

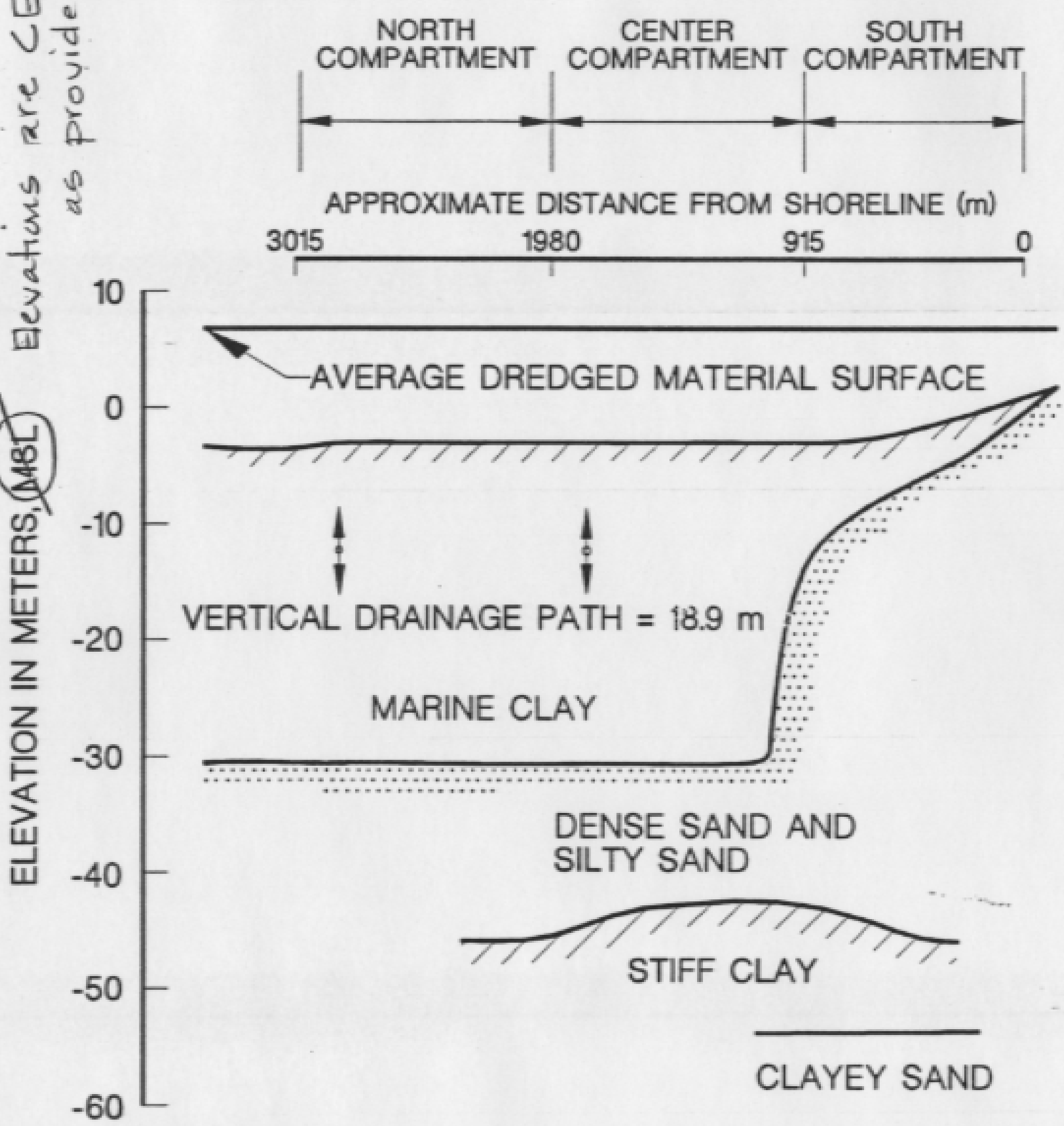


Figure 6. Generalized Subsurface Profile at Craney Island

quantity of dredged material would require substantial study. As a result, additional alternatives for increasing the storage capacity of the CIDMMA were sought.

7. Recently installed piezometers in the perimeter dikes at the CIDMMA revealed that large excess pore-water pressures exist in the marine clay. It can be seen from Figures 2 through 5 that the excess pore-water pressure levels in February 1991 exceed the ground surface elevation by 7.5 m in some locations. The dissipation of these excess pore-water pressures would result in substantial consolidation settlement, and thus increased storage capacity. In addition, the consolidation of the marine clay and dredged fill would cause a significant increase in the undrained shear strength of these materials. This would allow the perimeter dikes to be constructed to higher elevations without setbacks or stability berms.

8. The time required for 90 percent consolidation to occur can be estimated using the one-dimensional consolidation equation (Terzaghi and Peck 1967):

$$t_{90} = \frac{0.848 * H_{dr}2}{C_v} \qquad (1)$$

where H_{dr} is the maximum length of drainage path and C_v is the vertical coefficient of consolidation. This equation shows that the time required for consolidation is controlled by the coefficient of consolidation, that is, permeability, of the soil and the maximum drainage length that water must travel to exit the soil deposit. Since altering the permeability of a soil in situ is not practical, techniques were sought to decrease the drainage path to accelerate consolidation.

Use of Prefabricated Strip Drains to Increase Storage Capacity

9. Figure 6 shows the generalized subsurface profile at the CIDMMA. It can be seen that the average surface elevation of the dredged fill is +7.3 m MLW and the thickness of the dredged fill is about 10.5 m. The thickness of the marine clay is 27.5 m, and thus the combined thickness of the dredged fill and marine clay is 37.8 m. Since the site is doubly drained, the maximum vertical drainage path to either the top surface or the permeable sands underlying the marine clay is 18.9 m. It should be noted that the thickness of the marine clay varies throughout the site. For example, in the north compartment the marine clay is approximately 33.6 m thick, that is to El.-36.6 m, where

Places mudline @ -3.2m. Original was about -3.0m (-10ft) to -4.0m (-13ft). Also conflicts w/ depth of 13.5m (-6.2m) as stated on Pg 19. Latter figure agrees w/ Corps findings.

(Location was based on as far west as dredge pipe allowed. We were aware of channel and decided we best find out now if it was problem. See soil profile along entire perimeter dike. Channel is evident.)

an old river channel is located. The strip drain test section, described in Part III, was ~~inadvertently~~ located above this old stream channel, and thus the maximum vertical drainage in this area is approximately 22 m. For illustrative purposes the marine clay will be assumed to be 27.5 m thick. Recent piezocone penetration tests in the north compartment described herein suggest that the underlying dense sand and silty sand are permeable.

10. Figure 7 shows that the installation of vertical strip drains will result in radial flow as well as vertical flow. The strip drains are installed through a 0.6 m sand working platform into the dredged fill and marine clay. The spacing of the strip drains in the test section was 2.1 m, which will be described subsequently. Strip drains reduce the maximum drainage path to one-half of the strip drain spacing, that is, approximately 1.1 m, instead of one-half of the compressible layer thickness, that is, 18.9 m. This reduction in drainage path is extremely significant since the time rate of consolidation is a function of the length of drainage path squared (Equation 1). Therefore, the installation of vertical strip drains will result in a substantial reduction in the time required to consolidate the dredged fill and underlying marine clay. This will yield a rapid increase in storage capacity and undrained shear strength of the dredged fill and marine clay. ~~As a result, Stark (1992) proposed the use of vertical strip drains to increase the storage capacity, and thus service life, of the CIDMMA.~~ ← Original proposal dates back to 1980 (Dr. Robert Y.K. Cheng, PE) but didn't have equipment to install

11. It was proposed that strip drains be installed throughout the placement area and subsequently the perimeter dikes. The strip drains will consolidate the dredged fill and underlying marine clay in the placement area, which may permit future development of this site. Installing strip drains in only the perimeter dikes would be less expensive and may also result in more settlement because of the additional surcharge applied by the dikes. The strip drains would accelerate consolidation of the marine clay underlying the perimeter dikes and allow the dikes to be constructed to higher elevations. However, the strip drains would not consolidate the placement area and thus not reduce the elevation of the placement area. CE NAO is interested in consolidating the placement area because it may create opportunities for future development of the site, and possibly the construction of a new placement area. If consolidation of the 8.9 km^2 placement area is not economically feasible, strip drains ~~will~~ may be installed in only the perimeter dikes. ~~The second strip drain test section will be constructed parallel to the west perimeter dike to investigate the effect of strip drains, and thus consolidation, on the settlement and stability of the perimeter dikes.~~ Rip-rap and debris, long settled and submerged in perimeter dikes, may restrict or prohibit use of strip drains. A supplemental investigation by the PI under separate cover will investigate this option.

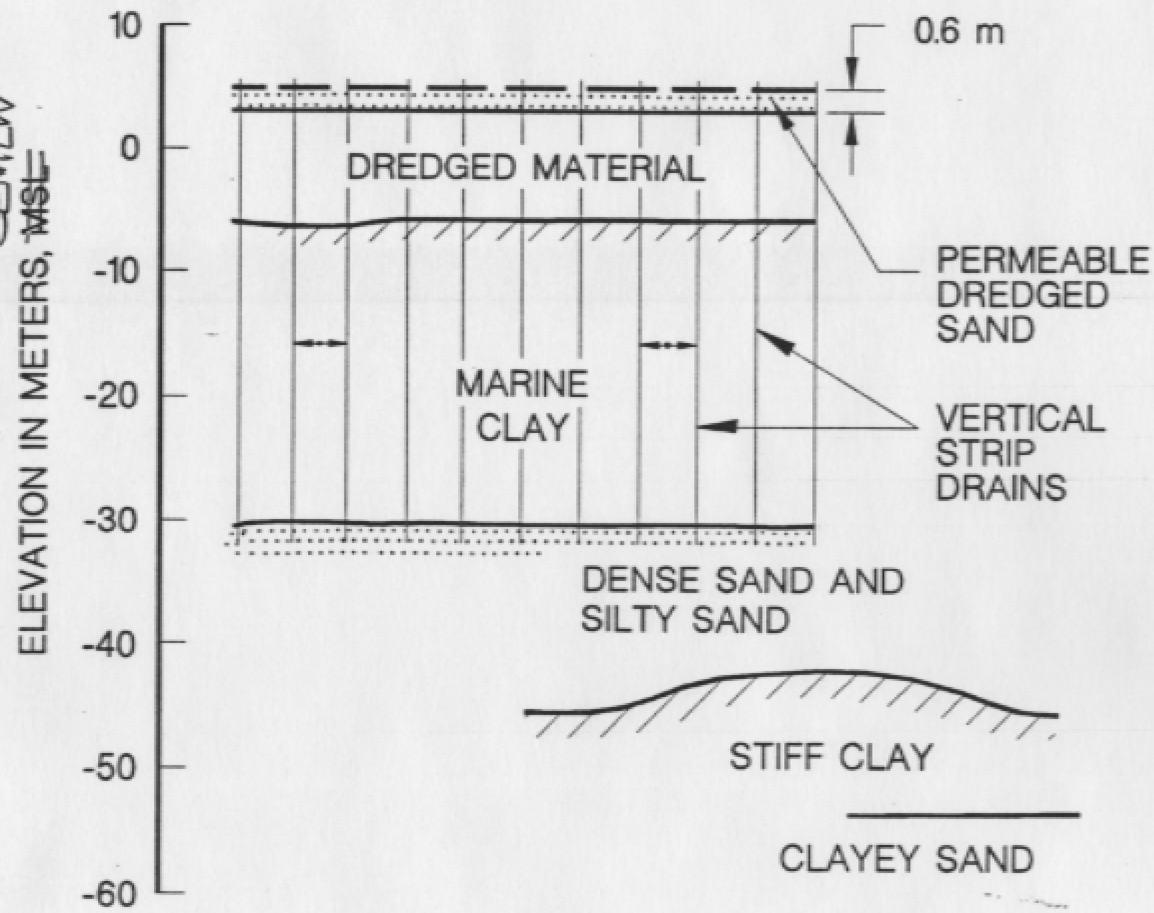


Figure 7. Radial Drainage Pattern
Using Vertical Strip Drains

PART II: VERTICAL STRIP DRAIN TECHNOLOGY

Background

12. Daniel E. Moran first proposed the use of vertical sand drains as a means for deep soil stabilization in 1925, and he received a U.S. patent on the concept in 1926 (Johnson, 1970). In the last 10 to 15 years, vertical strip drains have replaced conventional sand drains as the preferred method to accelerate the consolidation of soft cohesive soils. This is primarily due to ease of installation, higher flexibility and reliability, less environmental impact, and reduced cost of the strip drains. Most vertical strip drains are modeled after the cardboard strip drain developed by Kjellman in 1948 (a,b). Strip drains are band-shaped and have a rectangular cross section of approximately 10 cm wide and 0.4 cm to 0.5 cm thick. A plastic core with grooves, studs, or channels is surrounded by a filter fabric. The filter fabric is most commonly a nonwoven geotextile. The plastic core carries the excess pore-water to the ground surface and/or the underlying drainage layer, and the filter fabric keeps soils particles from entering the core. Vertical strip drains have been used to accelerate consolidation of soft cohesive soils in many projects throughout the United States, including the recent expansion of the Port of Los Angeles, the Seagirt project in Baltimore Harbor, construction of a dredge material containment area in the Delaware River near Wilmington, Delaware, and the New Bedford Superfund Site near New Bedford, Massachusetts.

13. Vertical strip drains are easily installed using equipment (Figure 8) that exerts a ground pressure as low as 20.7 kPa to 34.5 kPa. The installed cost of strip drains is usually $1.30 to $3.30 per lineal meter depending on the quantity of strip drains installed. In contrast, the installed cost of conventional sand drains is $11.50 to $21.30 per lineal meter. The time required for consolidation of the dredged fill and foundation clay is controlled by the spacing of the strip drains. Therefore, value engineering can be used to determine the optimal spacing of the drains to produce a certain increase in settlement, that is, storage capacity, in a specified time.

14. The strip drains arrive at the site in large rolls and are installed using a hollow mandrel. The end of the strip drain is threaded down the inside of the mandrel, which must be as long as the depth to which the strip drains are to be installed. At the bottom of the mandrel, the strip drain is threaded through a base plate and the end of the drain is inserted into the mandrel (Figure 9). The base plate is used to keep the strip drain at the bottom of the mandrel during installation, to prevent soil from entering the mandrel during the insertion process, and to keep the strip drain at the desired depth as the mandrel is withdrawn. When the mandrel is withdrawn from the ground, the strip drain is cut, and the process is repeated at the next location. This insertion cycle is rapid (1 to 5 minutes

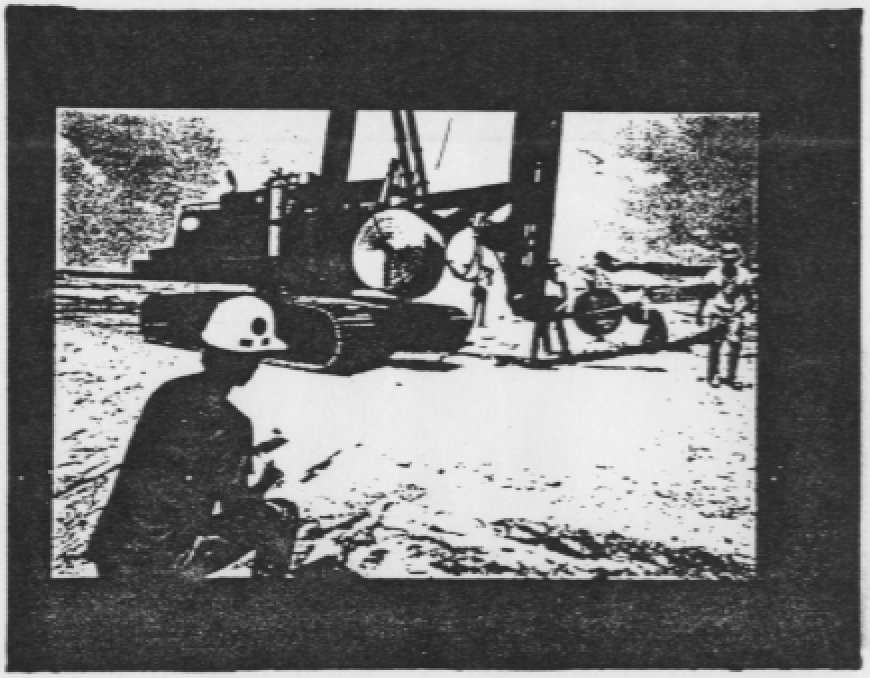


Figure 8. Typical Strip Drain Installation Equipment

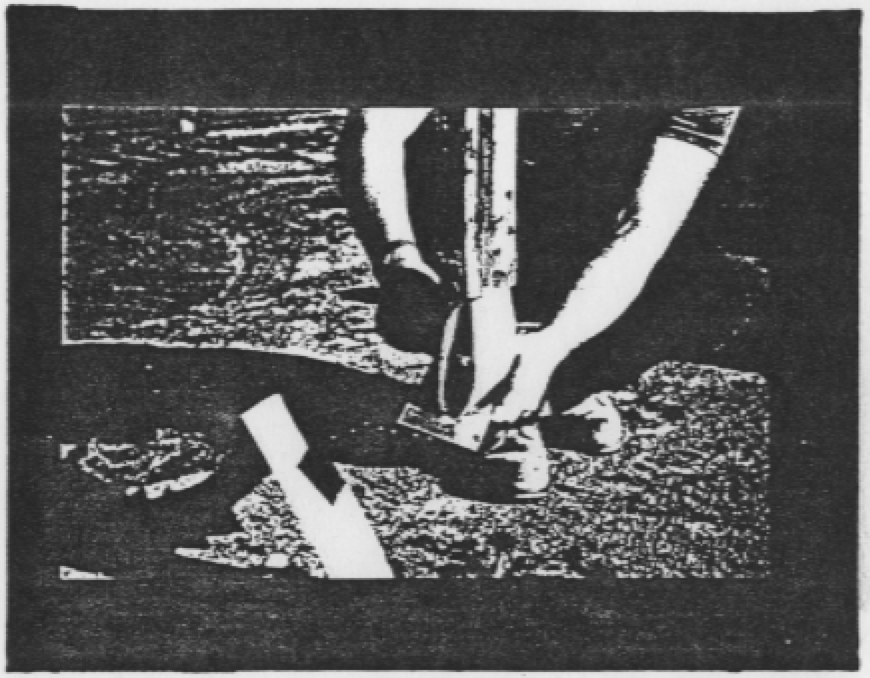


Figure 9. Strip Drain Installation Procedure

depending on insertion depth) and only strip drains, base plates, and a drain cutting tool are required.

15. At the CIDMMA it is anticipated that strip drains will be installed in one compartment while the other compartments are used for placement and desiccation. After the strip drains accelerate consolidation in the first compartment, this compartment will be used for placement while strip drains are installed in another compartment and the third compartment undergoes desiccation to support the strip drain equipment. Installation of strip drains will continue until strip drains have been installed in all three compartments.

16. The length of the strip drains will vary in each compartment with the longest strip drains (approximately 45 to 50 m) being installed in the north compartment where an old river channel is located. A number of contractors, e.g., Joiner (1991), have installed vertical strip drains to similar depths. For example, 36, 40, and 43 meter long strip drains were recently installed in New York, Utah, and Connecticut, respectively (Joiner, 1991). However, strip drains have never been installed in an active dredged material management area. As a result, existing strip drain equipment had to be modified to reduce the ground pressure less than 10.3 kPa to successfully operate on confined dredged material. In addition, a vertical strip drain length of 45 to 50 m would be close to the longest drain ever installed (less than 60 m). One of the main objectives of the strip drain test section was to investigate the feasibility of installing 45 to 50 m long strip drains from the surface of confined dredged material.

Vertical Strip Drain Design Theories

17. The design of vertical strip drains is generally based on the theoretical solution for radial consolidation developed by Barron (1948) in which the drains are assumed to be of infinite permeability. Hansbo (1979 and 1981) simplified Barron's solution and accounted for well resistance and the effects of smear due to drain installation (Figure 10). It can be seen that the degree of consolidation is a function of G and F(n,s). The variable G describes the effect of well resistance on the rate of consolidation and F(n,s) describes the effect of the smear zone. The well resistance is controlled by the influx of water to the strip drain and the flow along the drain. Therefore, G depends on the drain diameter, drain spacing, and the maximum drainage length of the strip drain. When strip drains are installed, the soil adjacent to the drain is disturbed and a smear zone is created. The extent of the smear zone depends on the installation procedure and the sensitivity of the soil. The overall effect of the smear zone is to reduce the permeability of the soil and slow the rate of consolidation.

Figure 10. Strip Drain Design Theory Presented by Hansbo (1981)

See pg 27 for t_{90} $= \pi\left(\frac{d_e}{2}\right)^2$

$$U_h = 1 - \exp\left(\frac{-8\ C_h\ t}{(d_e)^2 \times [F(n,s) + G]}\right) \tag{2}$$

$$F(n,s) = \ln\left(\frac{n}{s}\right) + \left(\frac{K_h}{K_s}\right)\ln(s) - 0.75 \tag{3}$$

$$G = 4\left(\frac{K_h}{K_w}\right)\left(\frac{l_m}{d_w}\right)^2 = \left(\frac{K_h\,(l_m)^2}{q_w}\right)\pi \tag{4}$$

where

U_h	= average degree of consolidation for radial flow;	
t	= time;	
C_h	= horizontal coefficient of consolidation;	7ft =
d_e	= sphere of influence of the strip drain (triangular pattern = 1.05S where S = strip drain spacing);	= 214 cm
d_w	= equivalent strip drain diameter = $\frac{2 * (b + l)}{\Pi}$	= 6.24 cm
b	= width of strip drain (typically 9.3 - 10 cm, used 9.45 cm);	
l	= thickness of strip drain (typically 0.3 - 0.4 cm, used 0.35 cm);	
n	= ratio of drain diameters = $\frac{d_e}{d_w}$	= 34
$F(n,s)$	= term describing smear zones;	
s	= ratio of smear zone diameter to drain diameter = $\frac{d_s}{d_w}$	
d_s	= outer radius of the smear zone;	
K_h	= horzontal coefficient of permeability of the undisturbed soil;	
K_s	= horizontal coefficient of permeability of the smeared soil;	
K_w	= coefficient of permeability of the strip drain;	
G	= term describing well resistance;	
q_w	= discharge capacity of strip drain = $\frac{\Pi}{4} K_w\, d_w^2$	
l_m	= maximum drainage length of strip drain.	

Figure 11. Strip Drain Design Theory Presented by Lo (1991)

NOT DEFINED.

$$U = 1 - \exp\left(-\left(\frac{8\,C_h}{d_e^2 \times [F(n,s) + G]} + \frac{4\,C_v}{H_{dr}^2}\right) t\right) \quad (5)$$

$$F(n,s) = \frac{n^2}{n^2-1} * \left[\ln\left(\frac{n}{s}\right) + \left(\frac{K_h}{K_s}\right)\ln(s) - 0.75\right] + \frac{s^2}{n^2-1}\left[1 - \left(\frac{s^2}{4n^2}\right)\right] + \frac{K_h}{K_s}\left(\frac{1}{n^2-1}\right)\left[\frac{(s^4-1)}{4n^2} - (s^2+1)\right] \quad (6)$$

.9513

$$G = 2.5\left(\frac{K_h}{K_w}\right)\left(\frac{l_m}{d_w}\right)^2 = 2\left(\frac{K_h\,(l_m)^2}{q_w}\right) \quad (7)$$

where

- U = average degree of consolidation for vertical and radial flow;
- t = time;
- C_h = horizontal coefficient of consolidation; See Eq 12 Pg 24
- C_v = vertical coefficient of consolidation; $C_v = \frac{C_h}{1.5}$ Pg 24.5.
- d_e = sphere of influence of the strip drain (triangular pattern = 1.05S where S = strip drain spacing);
- d_w = equivalent strip drain diameter = $\frac{2 * (b + l)}{\Pi}$
- b = width of strip drain (typically 9.3 - 10 cm, used 9.45 cm);
- l = thickness of strip drain (typically 0.3 - 0.4 cm, used 0.35 cm);
- n = ratio of drain diameters = $\frac{d_e}{d_w}$
- $F(n,s)$ = term describing smear zones;
- s = ratio of smear zone diameter to drain diameter = $\frac{d_s}{d_w} = 2$ See Pg 27
  - note $d_s = 2 d_w$
- d_s = outer radius of the smear zone;
- K_h = horzontal coefficient of permeability of the undisturbed soil; See Eq 11, Pg 24
- K_s = horizontal coefficient of permeability of the smeared soil; $k_s = 0.5 k_h$ See Pg 27
- K_w = coefficient of permeability of the strip drain;
- G = term describing well resistance;
- q_w = discharge capacity of strip drain = $\frac{\Pi}{4} K_w\, d_w^2$
- l_m = maximum drainage length of strip drain.
- H_{dr} = shortest drainage path

18. Yoshikuni and Nakanodo (1974) and Onoue (1988) have presented rigorous solutions to the radial flow problem that also account for the effects of smear and well resistance. However, these solutions are complicated, and thus difficult to use in practice. Lo (1991) simplified the rigorous solutions, which resulted in the solution shown in Figure 11. It should be noted that Zeng and Xie (1989) also developed a simplified solution that has a slightly different expression for the effect of well resistance.

19. A comparison of Figures 10 and 11 shows that the main differences between Lo's and Hansbo's solution are the expressions for G and F(n,s), and the effect of vertical flow on rate of consolidation. Review of several case histories has shown that the modifications presented by Lo (1991) provide excellent agreement with field case histories. The case histories also revealed that the importance of vertical drainage increases with increased drain spacing of strip drains.

PART III: FIELD TEST SECTION AND SUBSURFACE INVESTIGATION

Field Test Section Objectives and Layout

20. A 183 m by 122 m field test section was constructed, instrumented, and monitored to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and marine clay in the CIDDMA. The test section was constructed in the north compartment of the CIDMMA because of the presence of a well-developed desiccated crust (Figure 12). The north compartment also requires the longest drains, which will provide a good evaluation of the strip drain equipment and a comparison between measured and predicted effects of smear zone and well resistance. The vertical strip drain test section consists of two areas (Figure 13). The main area is 152 m by 122 m and is covered with a 0.6 meter thick sand blanket to promote surface drainage and support the installation equipment. The vertical strip drains were pushed through the sand blanket to the underlying dense sands (Figure 14). It can be seen that the bottom of the marine clay is located at El. -36,6 m MLW because of the presence of an old river channel.

21. The mobility test section is 30 m by 122 m and utilizes prefabricated horizontal drains to promote surface drainage. The main objective of the adjacent mobility section was to determine whether or not a sand blanket is required to install vertical strip drains throughout the remainder of the management area. As a result, the 15 cm to 30 cm thick desiccated crust in this area must support the installation equipment. The vertical strip drain equipment was required to exert a ground pressure that would enable the equipment to operate on the desiccated crust. It was anticipated that a maximum ground pressure less than or equal to 10.3 kPa would be required to operate on the crust. The vertical strip drains were installed in the mobility test section first so that the crust was not subjected to surface water from consolidation of the main test section. Installation of strip drains in the test section commenced on 23 December 1992 and terminated on 26 February 1993. – Dates conflict w/ Pgs 29 & 34.

Need to verify. I thought we featured sand blanket to allow contractor (at his request) to simply move along the entire row west – east and return.

22. The vertical strip drains in the mobility section are connected to horizontal strip drains on the ground surface. Each vertical strip drain is connected to a horizontal strip drain to promote drainage to the surrounding perimeter ditch. Figure 15 illustrates the connection of a vertical strip drain to a horizontal strip drain. Horizontal strip drains are being used to evaluate their effectiveness in conveying water from the test section to the surrounding perimeter trenches and their ease of installation. If the horizontal drains are effective and the desiccated crust can support the installation equipment, a sand blanket would not be required over the remainder of the site. In addition, the horizontal drains will promote drainage as future dredged material is placed in the CIDMMA.

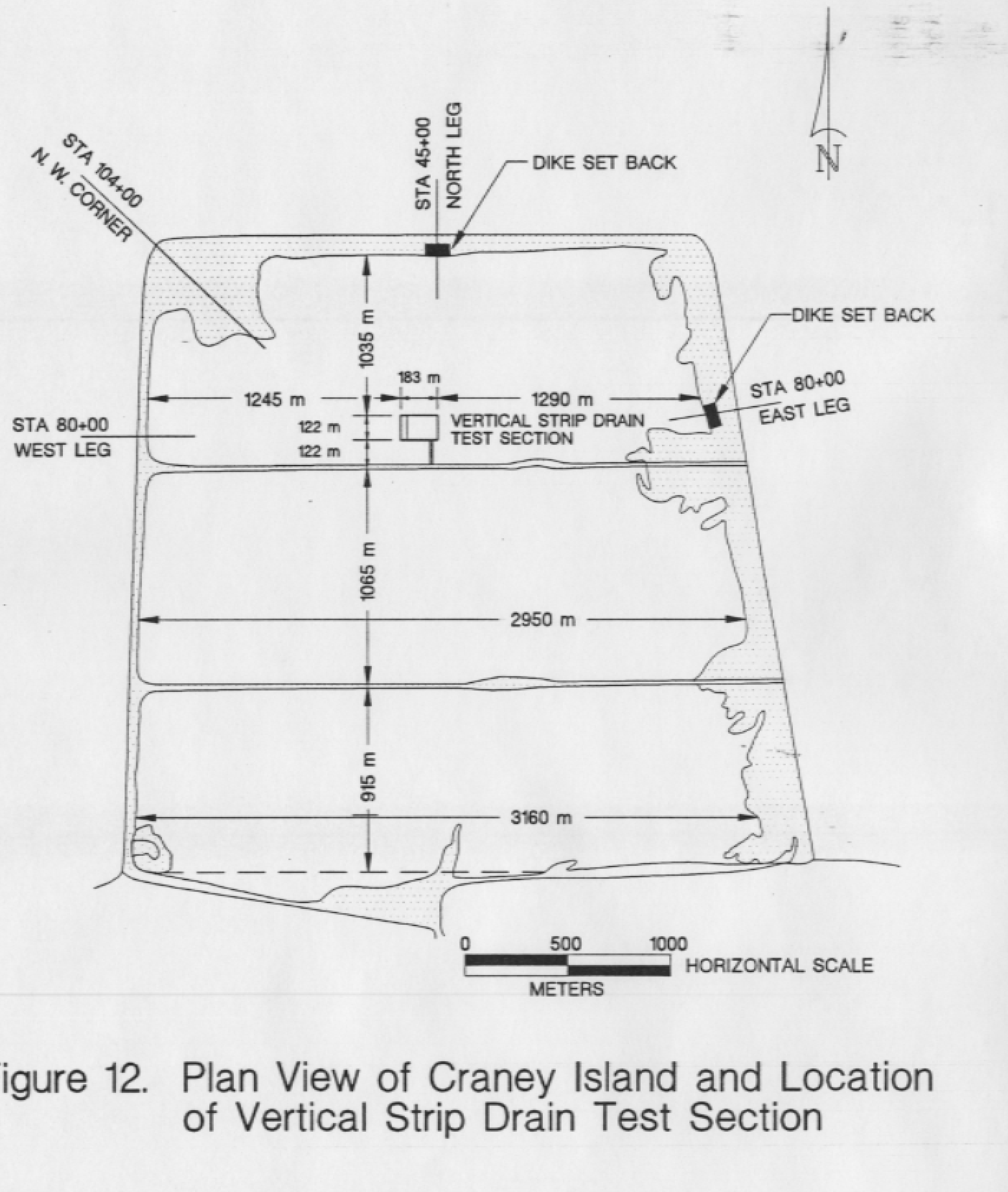


Figure 12. Plan View of Craney Island and Location of Vertical Strip Drain Test Section

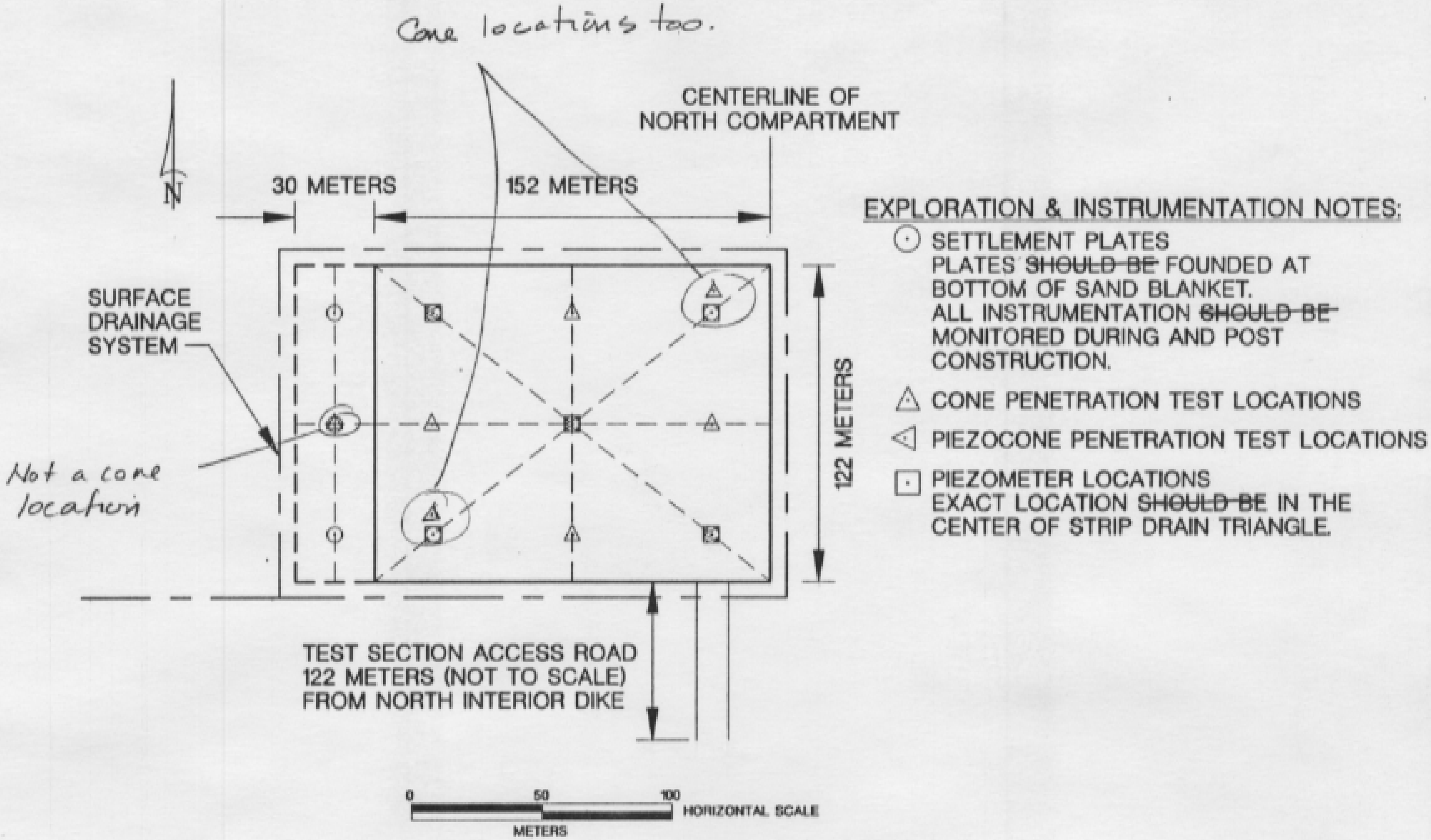


Figure 13. Plan View of Strip Drain Test Section and of Subsurface Exploration and Instrumentation at Craney Island

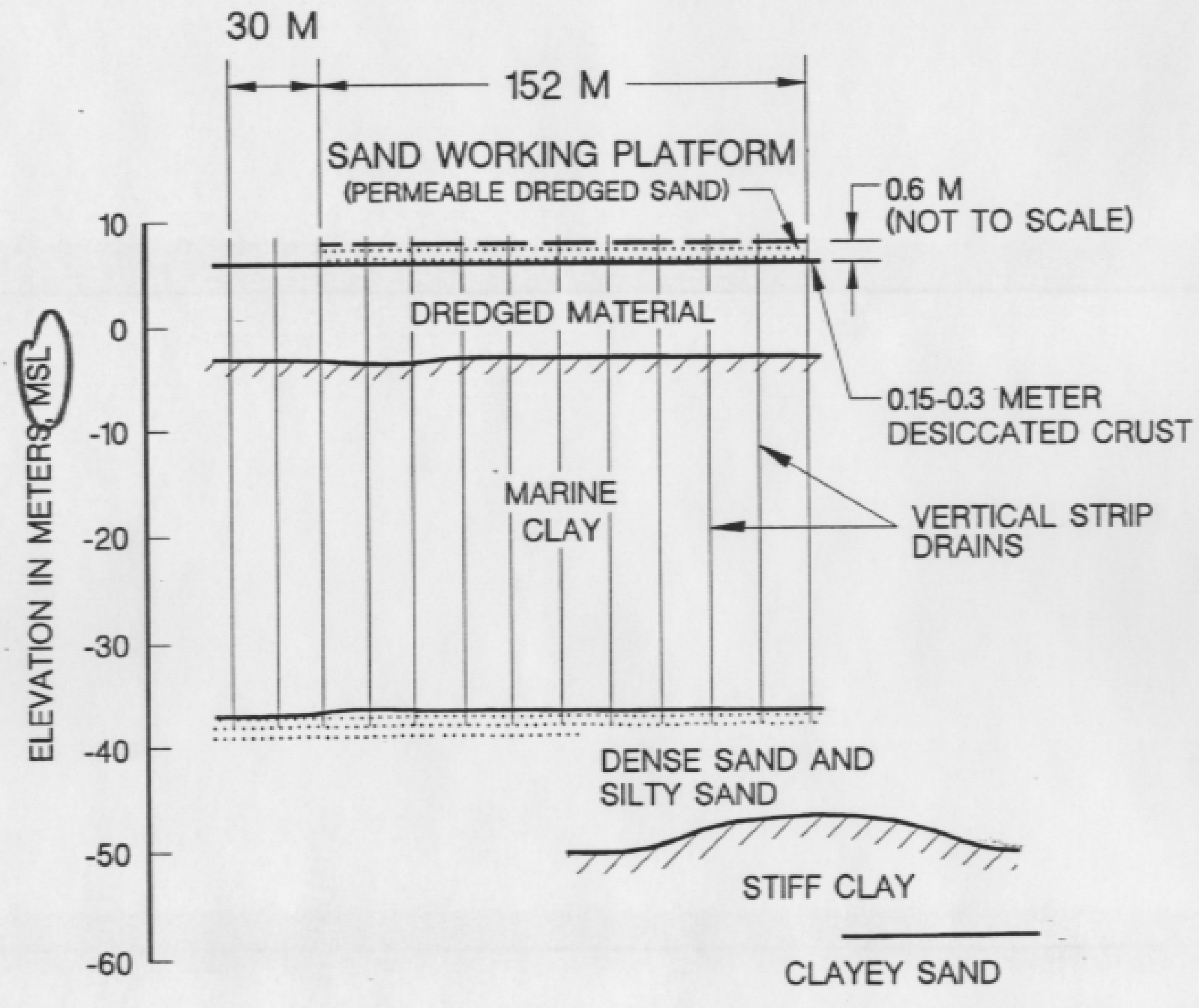


Figure 14. Generalized Subsurface Profile Near Vertical Strip Drain Test Section

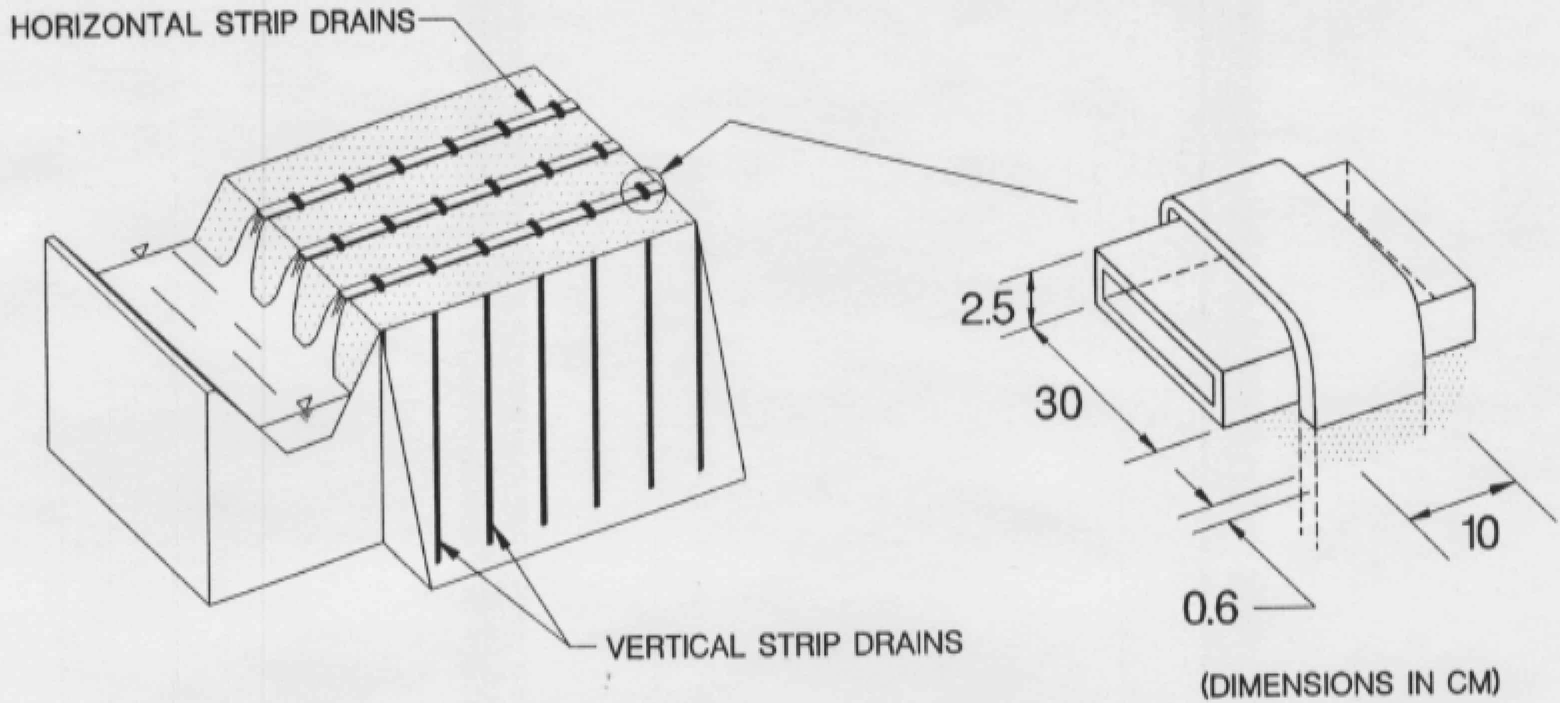


Figure 15. Horizontal and Vertical Strip Drain Installation Mobility Test Section at Craney Island

Horizontal strip drains were not installed in the main test section because the sand blanket will act as a drainage layer for future dredged fill.

Subsurface Investigation and Field Monitoring

23. A subsurface investigation was conducted in the area of the test section before strip drains were installed to aid interpretation of the consolidation settlements. The following is a list of the tests that were conducted to evaluate the subsurface conditions:

1.) Cone and piezocone penetration tests were ~~be~~ conducted to define the soil stratigraphy in the test section. These tests results were also used to estimate the magnitude and variability of the undrained shear strength, S_u, and the coefficient of consolidation with depth. Piezocone dissipation tests were used to determine the excess pore-water pressure condition prior to drain installation. Dissipation tests were conducted every 3 m to 6 m in the piezocone tests, and the results were also used to estimate the coefficient of consolidation.

2.) Pneumatic piezometers were installed at three locations (Figure 16) in the test section area using the cone penetration test equipment. A total of eleven piezometers were installed at varying depths at locations P-3, P-5, and P-7 (Figure 16) to aid in determining the variation of pore-water pressure with depth.

Never furnished to you.

3.) Field vane shear tests were conducted every 3 m to 6 m in a boring (B-1) that was drilled at the center of the test section (Figure 16). Water content samples were taken every 3 m in this boring. The field vane shear and moisture content tests were used with the cone penetration test results to estimate the magnitude and variability with depth of S_u, coefficient of consolidation, and initial void ratio.

4.) Eight settlement plates were installed throughout the test section to monitor the effectiveness of the strip drains (Figure 16). Three of the settlement plates are located in the mobility section and five in the main section. These settlement plates were installed and surveyed prior to installation of the strip drains. It should be noted that the plates were installed shortly after placement of the sand blanket. The settlement plates were surveyed periodically until the strip drains were installed. After installation of the strip drains, the plates were surveyed weekly for the first three and one-half months and are being surveyed monthly until consolidation is completed.

Surveyed weekly during installation, monthly after construction starting w/ March

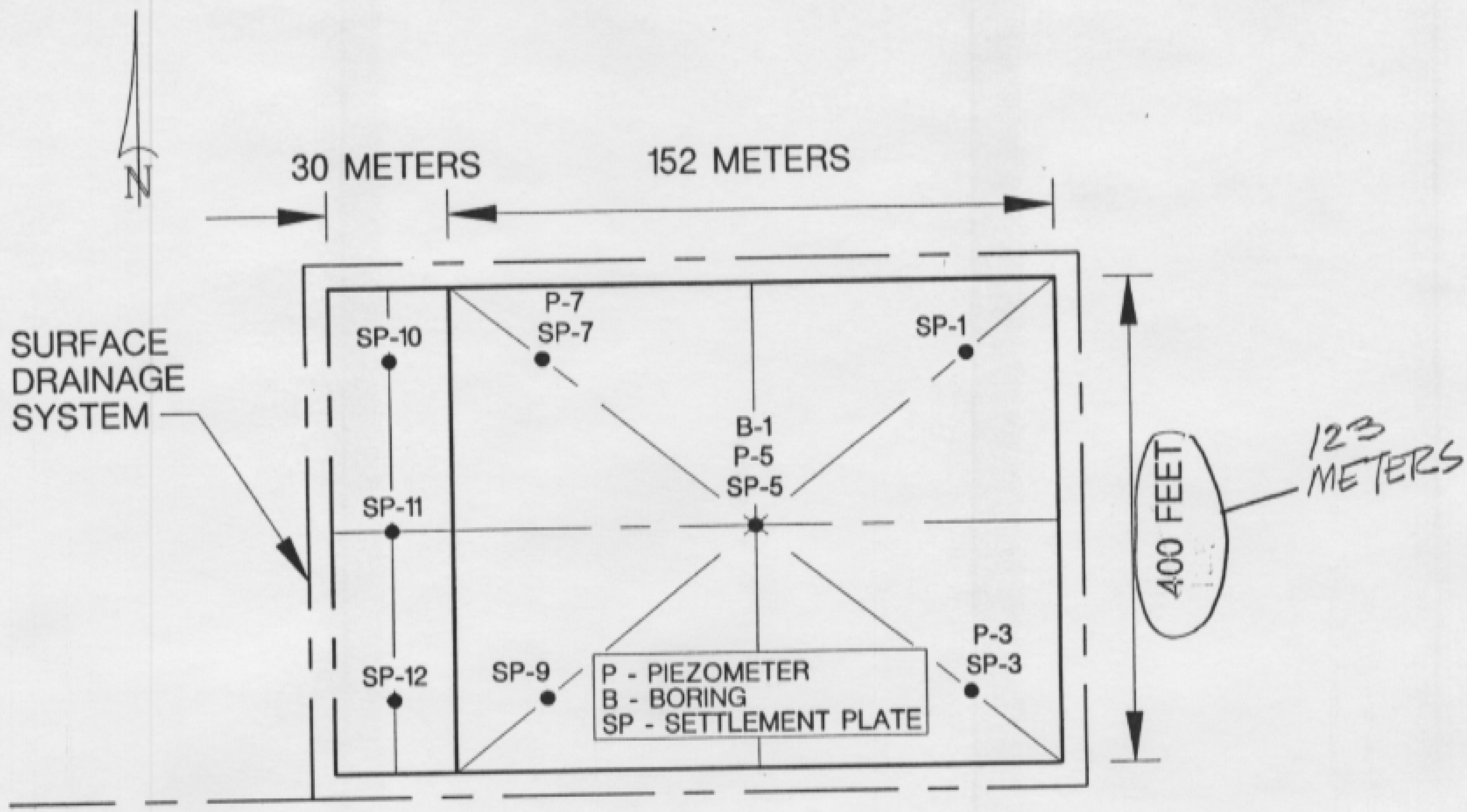


Figure 16. Location of Settlement Plates, Piezometers, and Boring in Test Section

Review

Initial Excess Pore-Water Pressures

24. Initial excess pore-water pressures were estimated from the installed piezometers and piezocone dissipation tests (Figure 17). The distribution of excess pore-water pressure clearly indicates that the marine clay is under-consolidated and the underlying dense sand is freely draining. It can be seen that the maximum excess pore-water pressure occurs at a depth between 15 m and 35 m or elevations -7.7 m MLW and -27.7 m MLW. It can be seen that the pore-water pressures at location P-7 (Figure 16) are lower than locations P-3 and P-5. It is anticipated that the higher pore pressures are due to the surcharge caused by the sand blanket and access road near location P-3 (Figure 13). Conversely, P-7 is located near the north edge of the test section where the sand blanket terminates.

25. The piezocone dissipation tests were conducted until the pore-water pressure measurement was constant. This was monitored using a microcomputer data acquisition system in the testing vehicle. Figure 18 shows the results of a dissipation test conducted at a depth of 27.5 m at the center of the main test area. It can be seen that approximately 80 minutes was required to achieve a constant pore-water pressure. These results are typical of all the piezocone tests, that is, approximately 70 to 80 minutes was required to obtain a constant pore-water pressure. However, plotting the dissipation data on a semi-logarithmic scale (Figure 19) revealed that the degree of consolidation at the end of the dissipation test is less than 90 to 95 percent. As a result, the semi-logarithmic dissipation relationship does not indicate the end of primary consolidation. This prevents the determination of the time at which 100 percent consolidation occurs, and thus the determination of the non-shear induced pore-water pressure. In summary, the piezocone data in Figure 17 overestimates the excess pore-water pressures because the pore-water pressures generated by cone insertion were not completely dissipated at the end of the test. Further evidence of this is that the effective overburden stress back-calculated using the dissipation test results is negative between depths of 15 m and 35 m. It is recommended that the data acquisition software be modified to present dissipation test results on a semi-logarithmic scale.

26. To clarify the initial excess pore-water pressure profile additional analytical techniques were utilized. The excess pore-water pressure was estimated from the undrained strength ratio, S_u divided by the preconsolidation pressure, σ_p'. The next section discusses the estimation of the undrained strength and an undrained strength ratio for the dredged fill and marine clay. The preconsolidation pressure was back-calculated using an undrained strength and the undrained strength ratio. The preconsolidation

Review

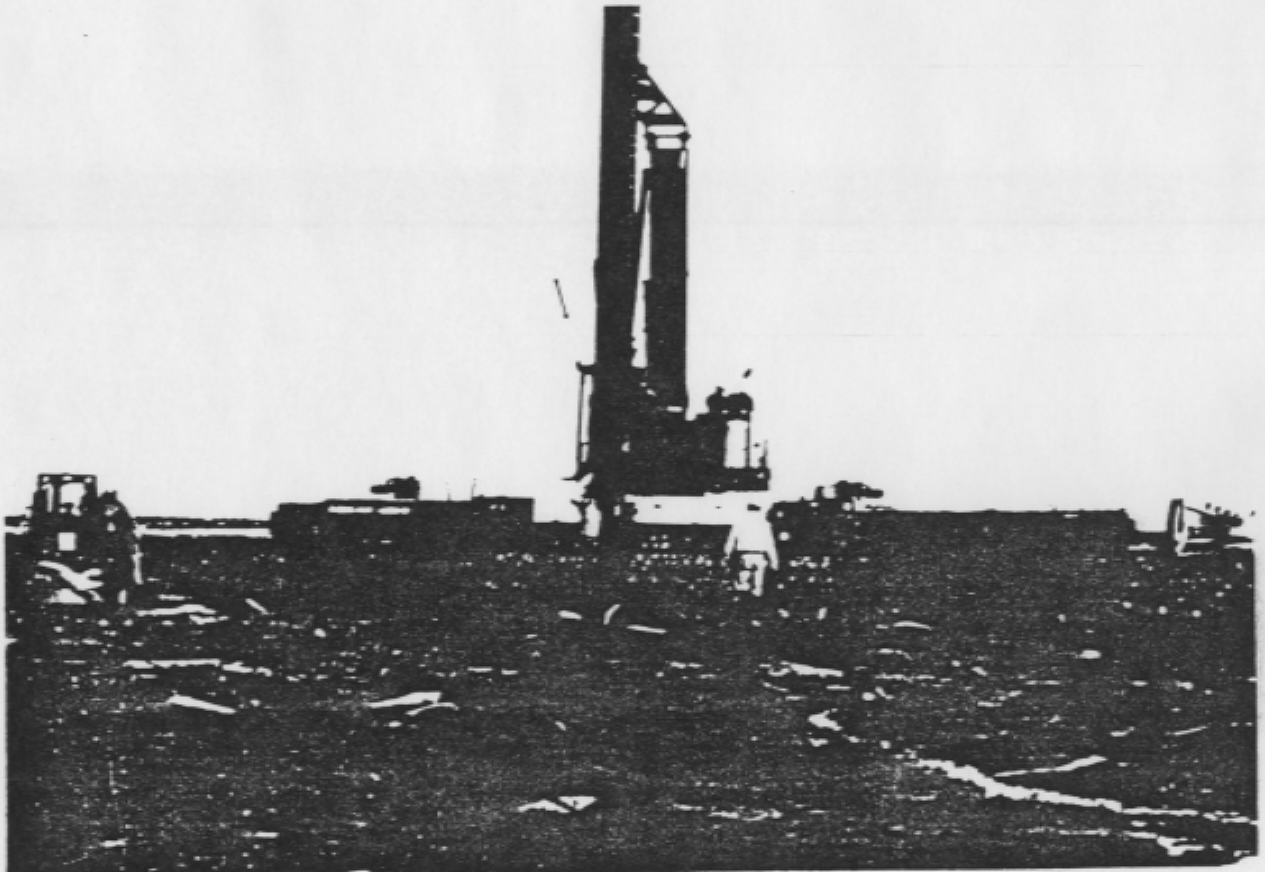
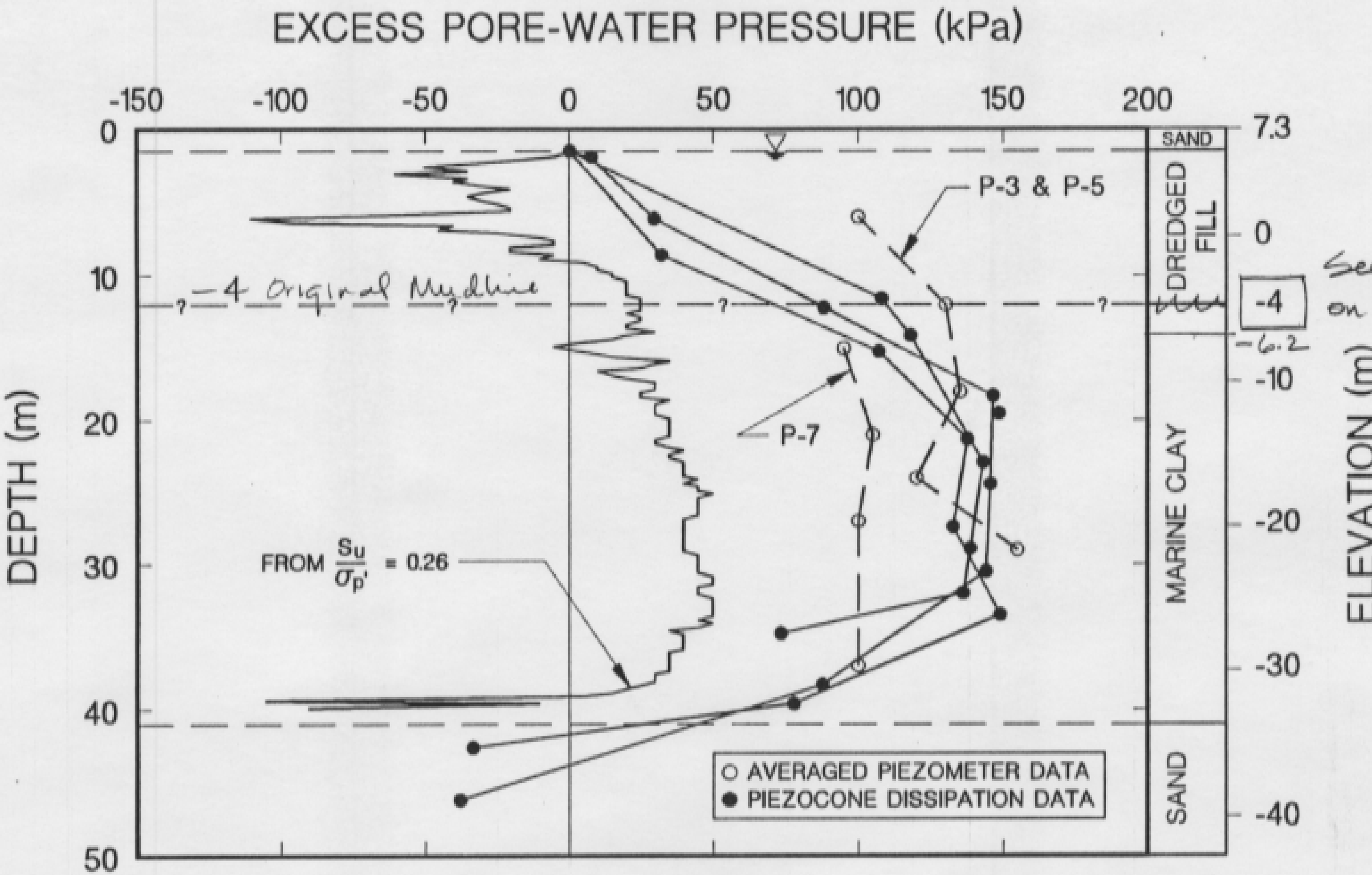


Figure 27. Installed Strip Drains and Pontoon Mounted Equipment

Out of Place. Follows Pg 28



See comment on Pg. 10 & 19. & Figure 20.

Figure 17. Excess Pore-Water Pressure Under Craney Island Strip Drain Test Section

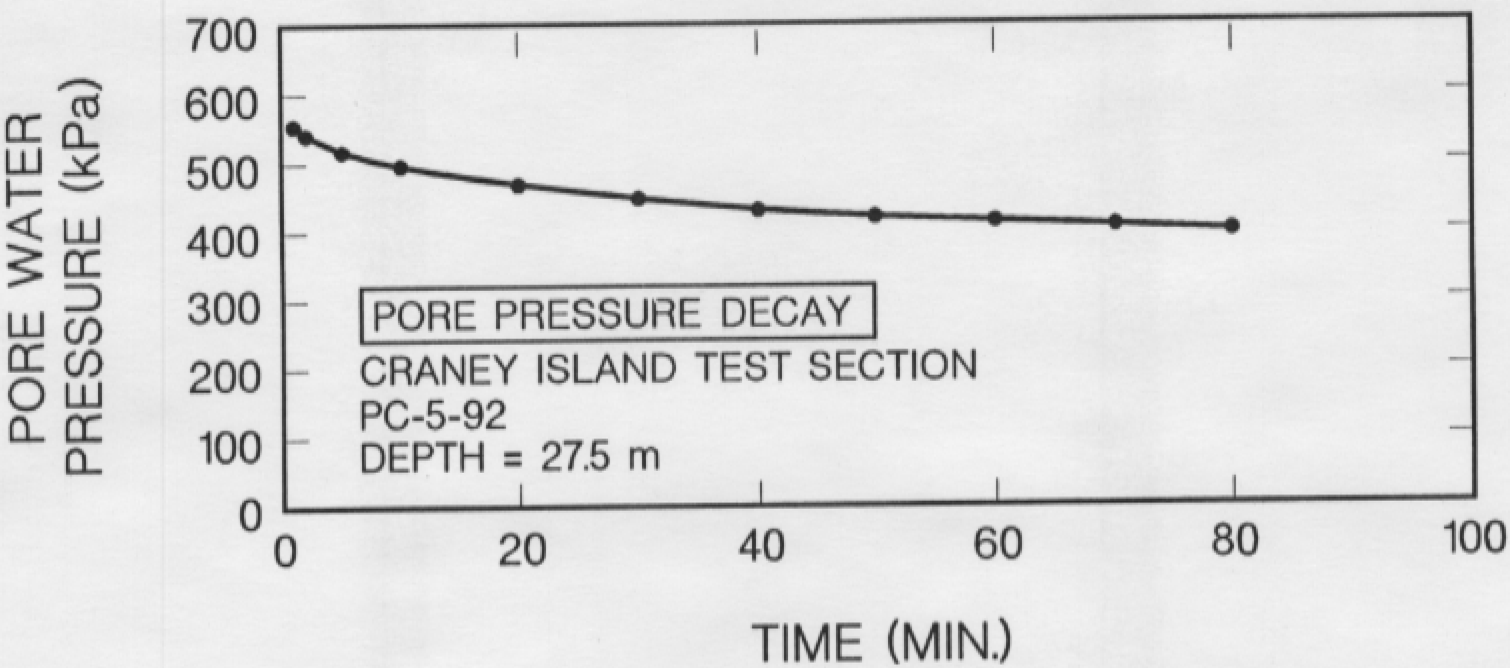


Figure 18. Typical Test Results from Piezocone Penetration Test

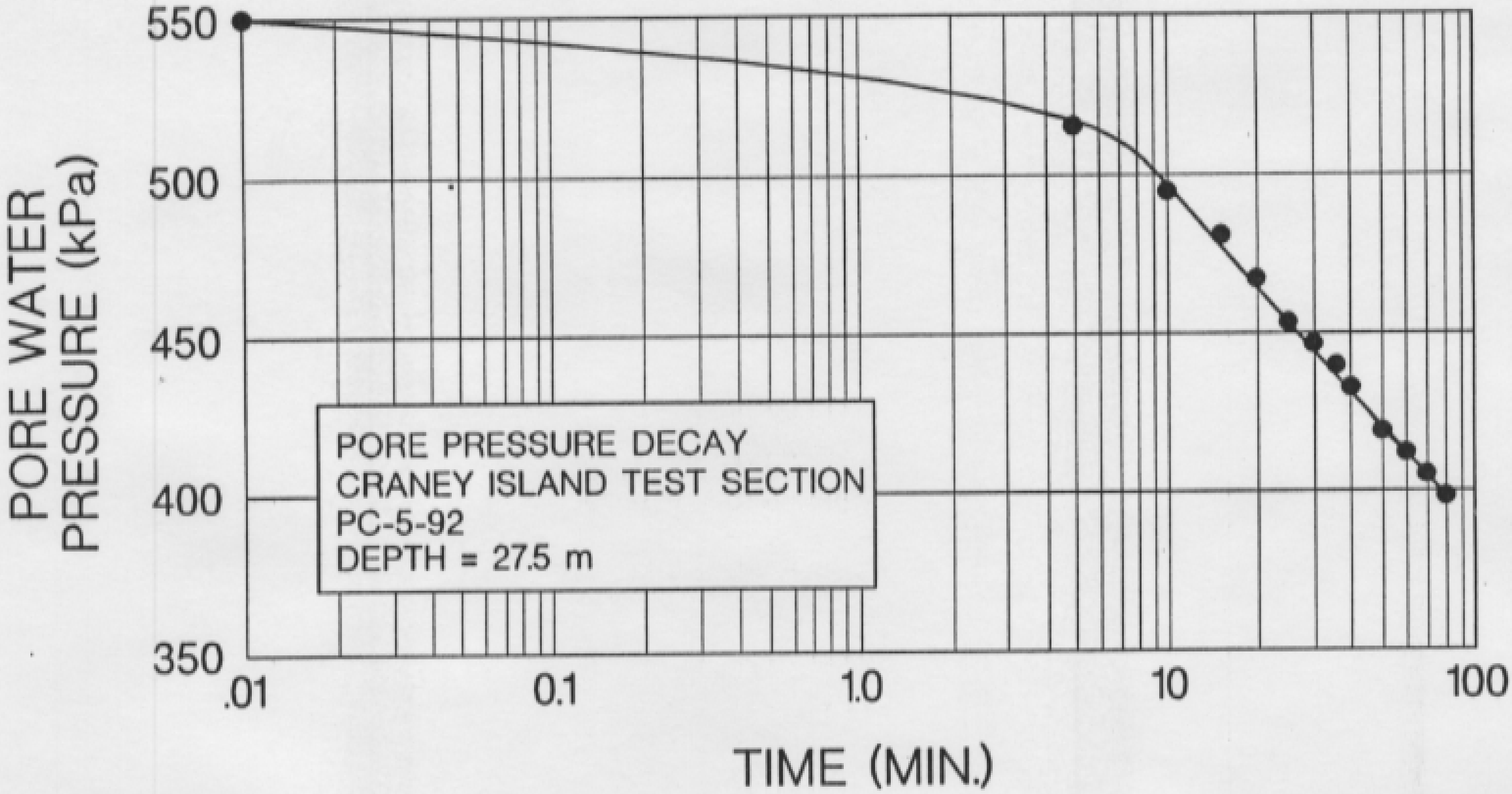


Figure 19. Semi-Logarithmic Presentation of Piezocone Test Results

PROGRESS REPORT (FIRST DRAFT)
U.S. Army Corps of Engineers Waterways Experiment Station
Corps of Engineers Contract No. DACW39-92-M-6666

STRIP DRAIN TEST SECTION IN CRANEY ISLAND DREDGED MATERIAL MANAGEMENT AREA

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August 8, 1993

Prepared for US Army Engineer, Waterways Experiment Station
Vicksburg, Mississippi 39180-6199

pressure was assumed to be equal to the current effective stress because the dredged fill and marine clay are under- or normally-consolidated. The excess pore-water pressure was estimated to be the difference between the calculated effective overburden stress and the effective stress after 100 percent consolidation. It can be seen from Figure 17 that the excess pore-water pressures estimated for an undrained strength ratio equal to 0.26 are less than the piezocone and piezometer data. The undrained shear strength was estimated from the tip resistance measured during the cone penetration tests, and thus the profile of excess pore-water pressure is continuous with depth.

See backside of front of Pg 17.

review

Initial Undrained Shear Strength

27. Consolidation of the dredged fill and clay foundation will result in a rapid increase in storage capacity and soil shear strength. The existing undrained shear strength profile in the test section was estimated using a number of techniques. The first technique described utilizes the tip resistance from cone penetration tests and the following equation:

$$S_u = \frac{q_c - \sigma}{N_k} \tag{8}$$

where q_c is the cone tip resistance, σ is the total overburden pressure, and N_k is an empirical cone factor. Empirical correlations of N_k have been developed using the results of field vane (Lunne and Kleven 1981 and Meigh 1987) and unconsolidated-undrained triaxial tests (Stark and Delashaw 1990). To differentiate the unconsolidated-undrained triaxial mode of failure, Stark and Delashaw (1990) denoted their cone factor N_{kuu}. Both correlations utilize plasticity index (PI) to estimate values of cone factor.

28. Table 1 presents the index properties of the marine clay at Craney Island. The statistical values of the index properties were determined from the results of 135 tests. Since the dredged material is similar to the foundation clay the same index properties were used for both deposits.

See comment Pg 16.

29. The field vane value of N_k was estimated for a PI of 41 ranges from 10 to 15, while the value of N_{kuu} ranges from 8 to 14. Since field vane shear test data are available, an average value of N_k equal to 12 was utilized in the analysis to facilitate comparison purposes. In addition, the average value of N_k is only slightly higher than the average N_{kuu} for this plasticity index. Figure 20 presents the variation of undrained shear

$\frac{S_u}{\sigma'} = 0.26$

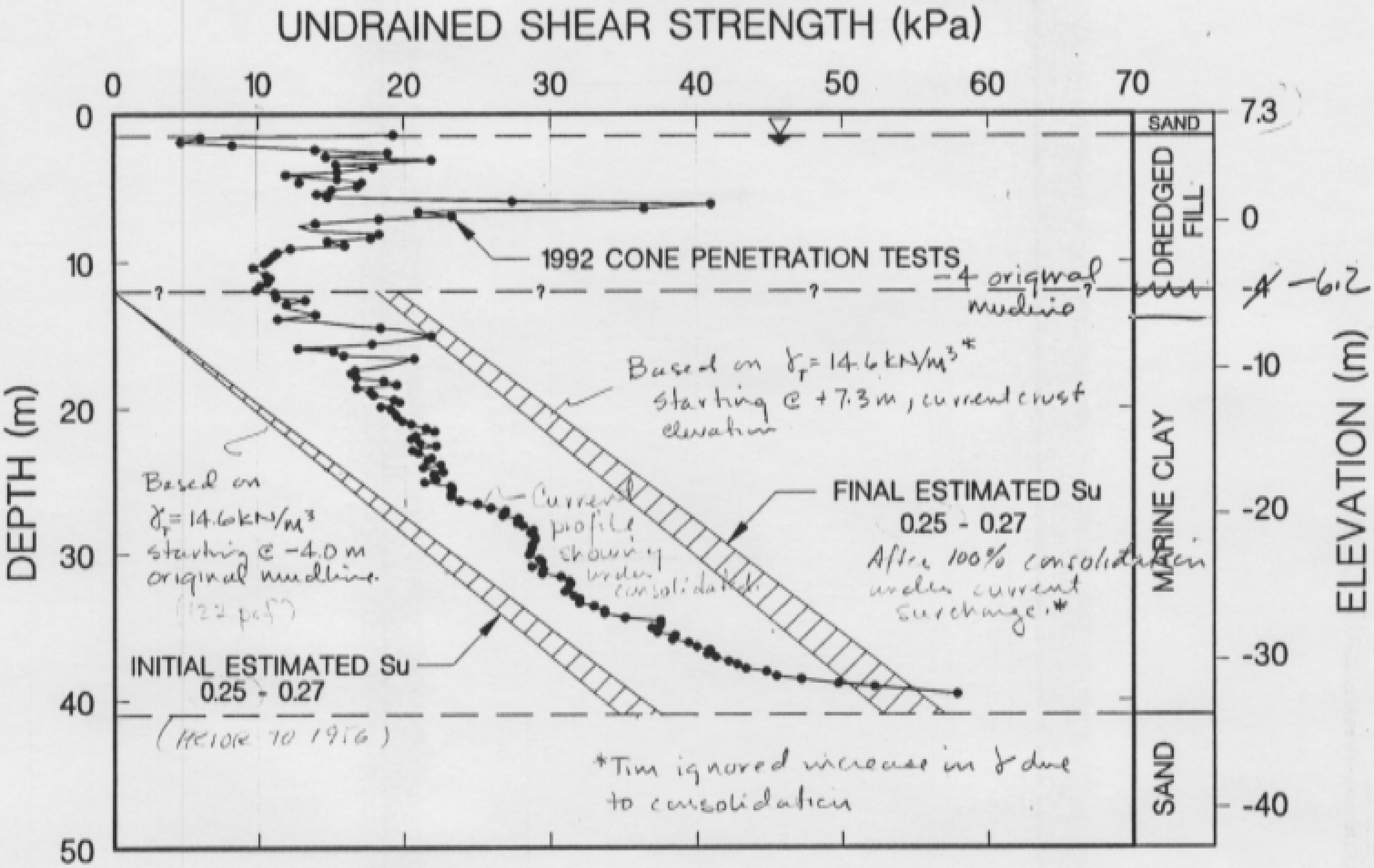


Figure 20. Undrained Shear Strength versus Depth in Craney Island Strip Drain Test Section

* For simplicity, assume γ_T = 14.6 kN/m3, same as unit weight used for initial profile. Actually need to comput what γ_T would be after 100% consolidation under self weight. Based on 1992 data, γ_T has increased to 14.9 kN/m³. (Using σ' from CPT's, presently U≈50%. γ_T at 100% would be in order of 15.2 kN/m³ See separate calcs)

strength with depth using N_k equal to 12. Each data point corresponds to a calculation of S_u using Equation (2) 8, the appropriate total stress, and a value of N_k equal to 12.

Table 1. Summary of Index Properties of Foundation Soil (after Ishibashi et al. 1992)

	Liquid Limit	Plastic Limit	Plasticity Index	Clay Size Fraction	Specific Gravity of Solids
AVERAGE	70.7	29.3	41.4	94.4	2.71
STANDARD. DEVIATION.	14.7	4.88	12.3	7.25	0.04
COEFFICIENT OF VARIATION	0.21	0.17	0.3	0.04	0.02

30. Figure 20 presents the variation in S_u with depth estimated from cone penetration test results and several interesting facts can be ascertained from the profile. First, the dredged material contains many sand and/or silt seams. This explains the lack of large excess pore-water pressures measured in the piezocone dissipation tests and piezometers in the dredged fill. The dredged fill is probably undergoing self-weight consolidation and the excess pore-water pressures are being dissipated by the sand/silt seams. Based on this conclusion, the majority of the consolidation settlement measured in the test section is attributed to consolidation of the marine clay. The dredged fill appears to be undergoing self-weight consolidation and acting as a surcharge for the marine clay.

31. Secondly, the marine clay appears to be under- or normally-consolidated. This is evident by the smoothness of the S_u profile and slight increase in S_u with depth. In addition, it also appears that the sand underlying the marine clay is free-draining because the values of S_u increase near the bottom of the marine clay. In fact, the value of S_u near the bottom of the marine clay corresponds to the effective stress at 100 percent consolidation.

Conflicts w/ Pg 10.

32. Thirdly, the interface between the dredged fill and marine clay appears to be located at a depth of approximately 13.5 m or El. -6.2 m MLW. Craney Island was constructed in approximately 3 to 4 m of water. Therefore, it appears that the dredged fill and marine clay interface has subsided 2.2 m to 3.2 m since 1957.

Undrained Strength Ratio

33. The undrained strength ratio of the marine clay was estimated from historic field vane shear (FV), unconsolidated-undrained triaxial (UU), unconfined compression (UC), and isotropically consolidated-undrained (CU) triaxial tests collected since 1948. Table 2 summarizes the values of

undrained strength ratio (S_u/σ_p') estimated from the tests reported in the General Design Memorandums (U.S. Army 1949 and 1986) for Craney Island. Table 2 reveals that the undrained strength ratio ranges from 0.24 to 0.28. The presence of gas in the dredged fill and marine clay complicates the collection and testing of undisturbed specimens. As a result, the most reliable measure of the in-situ undrained strength ratio is obtained using a historic field test, such as the field vane shear or cone penetration test. It can be seen that the field vane shear test yielded an average undrained strength ratio of 0.26.

34. For comparison purposes, the undrained strength ratio was estimated from published correlations, such as Figure 21. The undrained strength ratio for an average plasticity index of 41 and the field vane mode of shear ranges from 0.25 to 0.27 with an average of approximately 0.26. In summary, a range of undrained strength ratio of 0.25 to 0.27 was used in Figure 20 and an average ratio of 0.26 was used in the analysis described herein.

Table 2. Undrained Strength Ratios for Marine Clay from Various Test Methods (after Ishibashi et al. 1992)

TEST METHOD	NO. OF MEASUREMENTS	AVERAGE S_u/σ_p'	STANDARD DEVIATION	COEFFICIENT OF VARIATION
FV	102	0.26	0.04	0.16
UU	55	0.24 *	0.13	0.46
UC	56	0.28	0.16	0.55
CU	10	0.27	0.05	0.17

* Extreme Values of Undrained Strength Ratio Higher than 0.7 were Removed

35. The undrained strength ratio (Ishibashi et al. 1992) was used to estimate the S_u profile using a total ~~measured~~ unit weight of 14.6 kN/m^3 and by assuming that the marine clay is normally consolidated. Figure 20 shows the initial estimated S_u profile, which corresponds to the S_u profile before Craney Island was constructed, that is, prior to 1956. As a result, the range of S_u was estimated using an S_u/σ_p' equal to 0.25 to 0.27 ~~and a normally~~ ? consolidated marine clay starting at an original mudline @ ~~a depth of 13.5 m or~~ El. -4.1 m MLW. A comparison of the initial estimated profile and the profile estimated from the 1992 cone penetration tests reveals that only a small amount of consolidation, and thus shear strength increase, has occurred between 1956 and 1992.

36. Undrained strength ratios of 0.25 and 0.27 were also used to estimate the increase in S_u that will result from installation of prefabricated strip drains, and thus 100

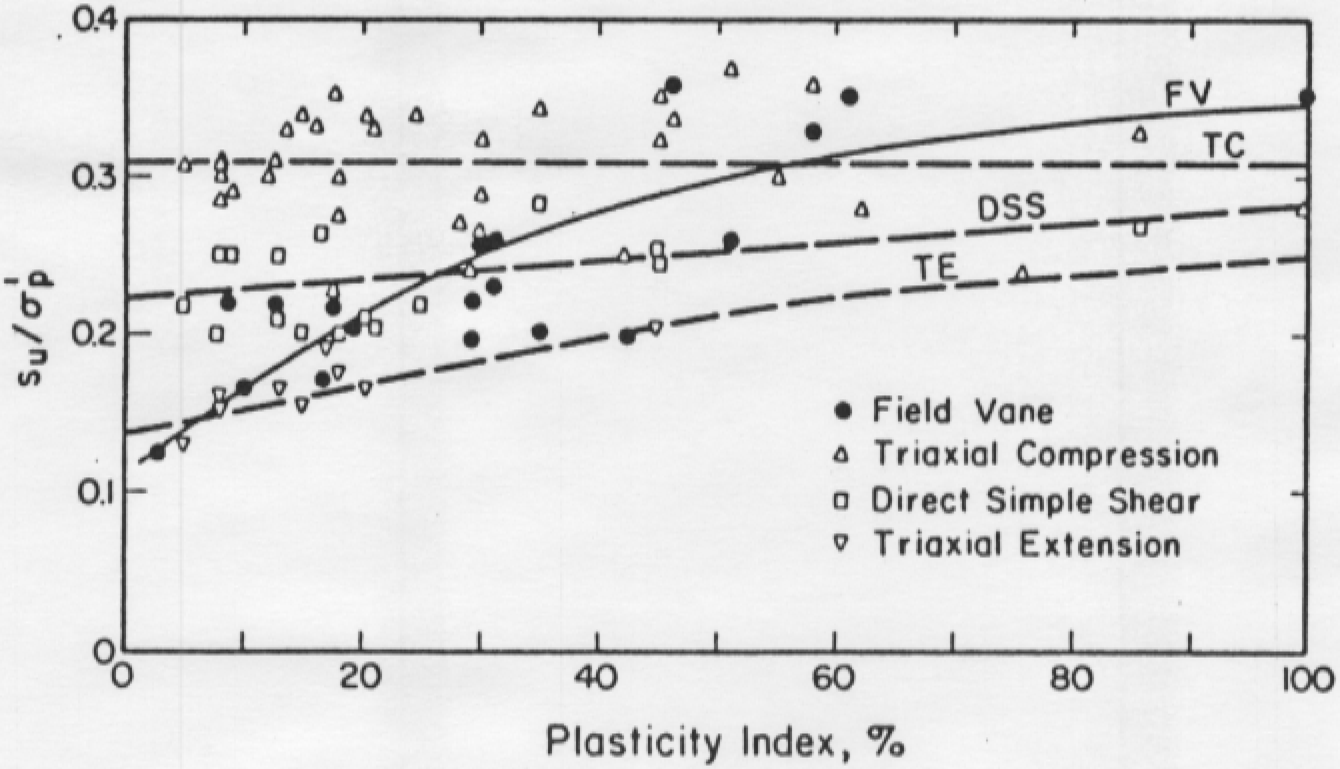


Figure 21. Values of Undrained Strength Ratio from Laboratory and Field Tests (From Mesri, 1989)

percent consolidation of the marine clay. Figure 20 shows the final estimated S_u profile, which was estimated assuming the dredged fill and marine clay are normally consolidated and the dredged fill surface is at El. +7.3 m MLW. It can be seen that the marine clay will probably undergo a substantial increase in undrained shear strength due to consolidation. For example, at a depth of 30 m the initial, 1992, and final values of S_u are 24, 28, and 44 kPa respectively. Therefore, between 1956 and 1992 a strength gain of only 16 to 17 percent occurred in the marine clay because of the slow rate of consolidation. The use of strip drains decreases the drainage path and accelerates consolidation. As a result, an increase in S_u of 85 to 90 percent is expected by early 1994 when the marine clay achieves a degree of consolidation of approximately 100 percent. Cone penetration tests will be conducted in early 1994 to verify the increase in S_u.

CHECK 1994 TEST DATA IN DETAIL

37. In summary, the installation of vertical strip drains will cause a substantial increase in undrained shear strength. ~~This increase should allow the perimeter dikes to be constructed to higher elevations without dike setbacks or stability berms. A study is being initiated by the Principal Investigator to determine how high the west perimeter dike (the least stable dike with respect to foundation stability) can be raised after 100 percent consolidation is achieved~~.

CONCLUSIONS NOT SUPPORTED BY POST-TEST

Initial Water Content Profile

38. One boring was drilled at the center of the test section (Figure 16) in March 1993. Samples were obtained with a piston tube sampler from the boring every three meters to a depth of approximately 34 meters. Unfortunately, the piston system was unable to retain ~~no~~ samples ~~were obtained~~ between the depths of 12 and 22 meters (Figure 22) and the samples were lost during retrieval. ~~In these samples gas pressure appeared to extrude the samples from the shelby tubes~~. Natural water contents (W_N) were determined for the recovered samples and are compared to the plastic (W_P) and liquid limits (W_L) of the samples (Figure 22). It can be seen that the water contents of the dredged fill and marine clay are at or near the liquid limit. A water content near the liquid limit indicates a soft to liquid consistency. Also shown in Figure 22 are water contents measured prior to construction of Craney Island (U.S. Army 1949). It can be seen that the 1956 and 1993 profiles are similar, especially below a depth of 20 m to 35 m. This also indicates that minimal consolidation has occurred in the marine clay since 1956.

Initial Void Ratio Profile

39. Void ratios were determined for the samples obtained from the boring at the center of the test section. The void ratios (e) were estimated using a degree of saturation (S) of 100 percent, a specific gravity of soil mass (G_s) equal to 2.71, and the following

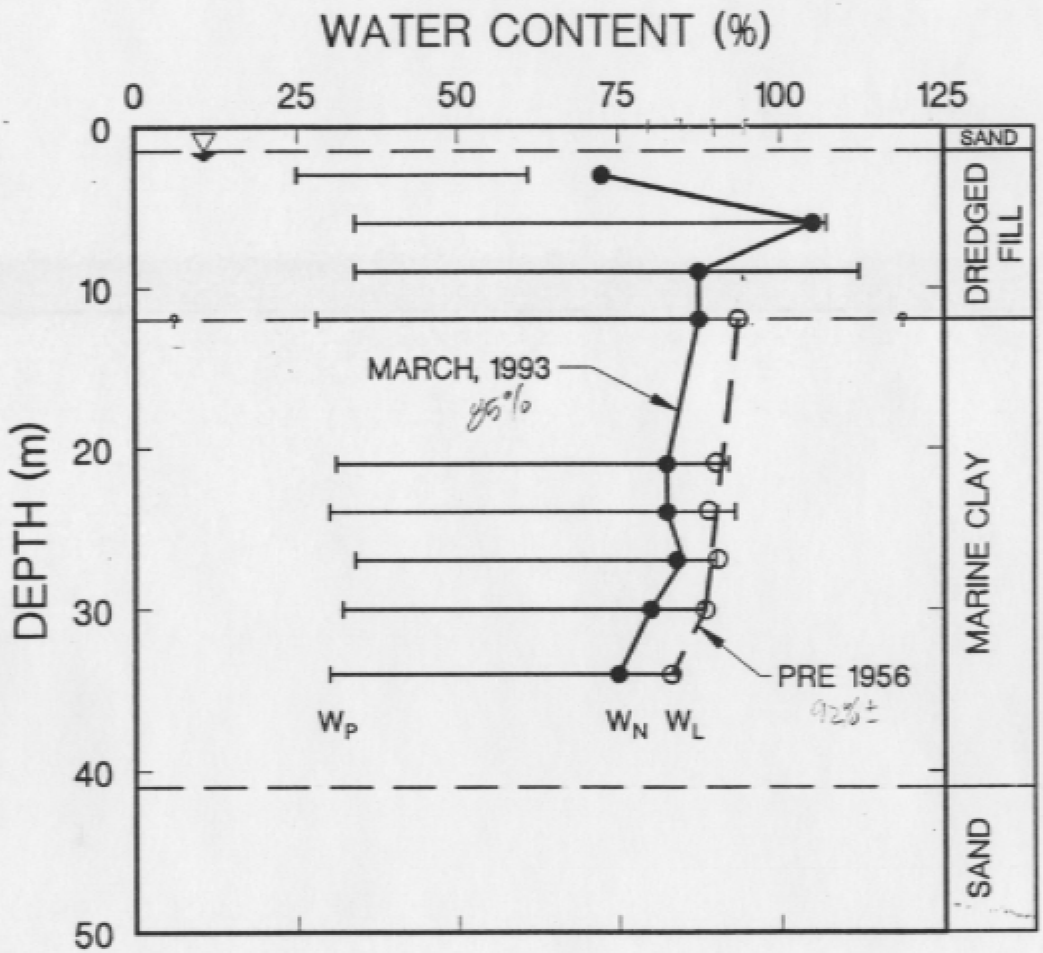


Figure 22. Water Content versus Depth in the Craney Island Strip Drain Test Section

equation:

$$S^*e = G_s{}^*W \tag{9}$$

See Pg 24. You used e = 2 to compute Cn.

It can be seen from Figure 23 that the average void ratio of the marine clay is approximately 2.5. The dredged fill exhibited void ratios of 2 to 3 and considerably more scatter than the marine clay. Figure 23 also presents a pre-1956 void ratio profile estimated from the water content test results (Figure 22) obtained from (U.S. Army 1949). It can be seen that the void ratio of the marine clay has not undergone a substantial decrease from 1956 to 1993. The void ratio after 100 percent consolidation was estimated using a range of values for the compression index (C_c). The range of C_c (0.549 to 1.362) was estimated using data from oedometer tests described in the next section. ~~It can be seen that the void ratio of the marine clay would be reduced to between 1.5 and 2.0 if 100 percent consolidation is achieved.~~

suggest delete material belongs in Pg 33; here [illegible] it is [illegible] that clearly [illegible] w/o more detailed explanation [illegible] developed.

40. Figure 24 illustrates the change in void ratio and bulk density that typically occurs in dredged material. This relationship between water content and void ratio was developed using a liquid limit of 71, a plasticity index of 41, and a specific gravity of soil mass equal to 2.71 (Table 1) for the Craney Island dredged material. Dredged material usually enters a management area from a discharge pipe at a void ratio of 10 to 20, or a bulk density of 1.15 to 1.08 kg/liter, respectively. With surface management, the decant water content can be reached at a void ratio of 4 to 5. Surface management includes pulling weir boards to decant the surface water after sedimentation of the dredged material is complete. With additional surface management and some desiccation, the water content can be reduced to the liquid limit (void ratio of 2 to 3) and the saturation limit (void ratio of 1 to 2), respectively. Additional desiccation can reduce the degree of saturation (S) below 100 percent and a water content corresponding to the plastic limit may be obtained. The lowest water content that can be obtained through desiccation is the shrinkage limit, which corresponds to a void ratio of less than 1.0. Review

41. From Figure 23 the void ratio of the dredged fill ranges from 2 to 3. It can be seen from Figure 24 that this void ratio corresponds approximately to the liquid limit. Therefore, the surface management program at the CIDMMA has been effective in reducing the void ratio to a water content that corresponds to approximately the liquid limit. However, additional decreases in void ratio could occur if consolidation is promoted.

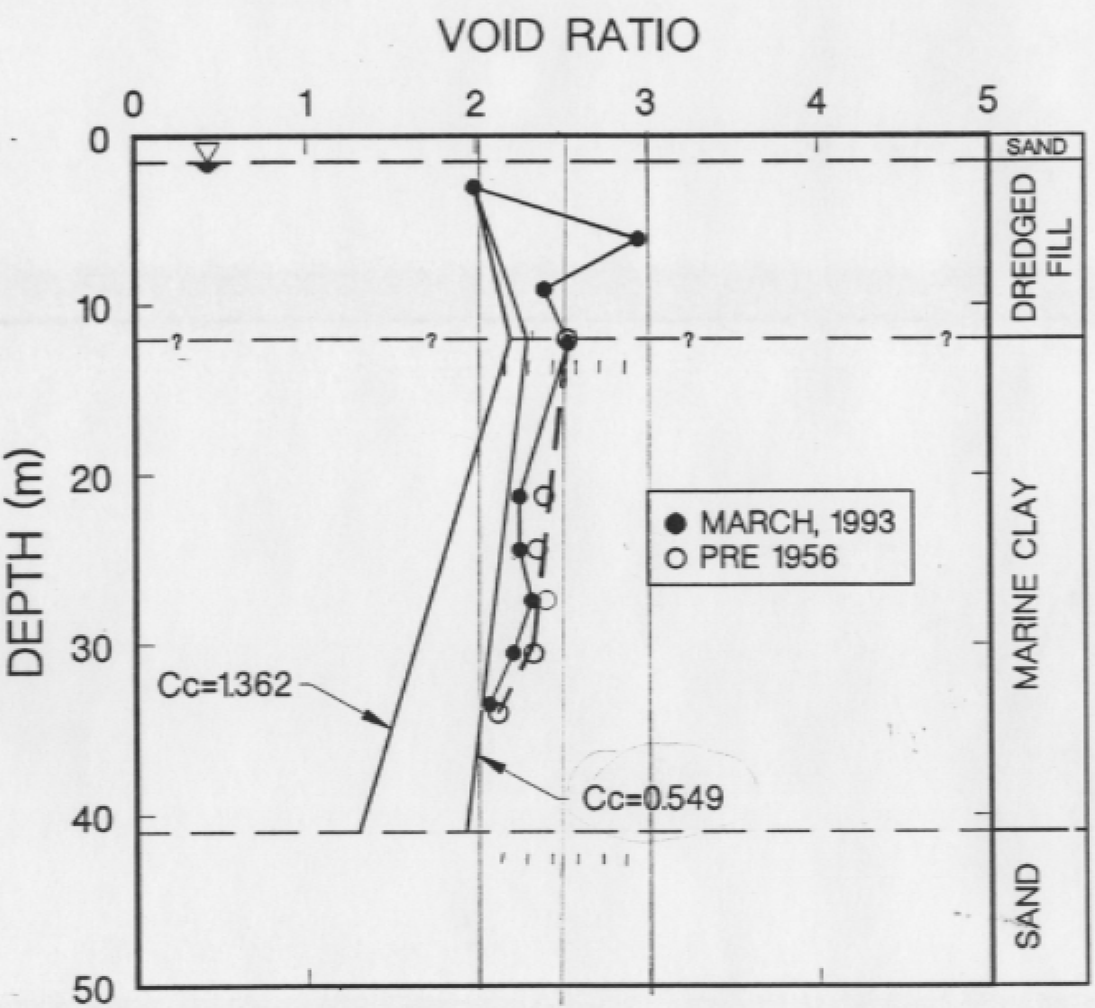


Figure 23. Predicted Void Ratio versus Depth in the Craney Island Strip Drain Test Section after 100% consolidation

Need to explain how developed straight line plots.

Current effective stress based on CPT data? $\sigma' = \frac{Su}{0.26}$

Final effective stress based on what?

$\sigma'_f = \sigma_T - u_{static}$

$\sigma_T = 14.6 \text{ kN/m}^3 \text{ h}$ —— Using 1948 γ_t probably underestimate γ_t @ 100% by 0.5 kPa/m³.

u_{static} on present WT.

$\Delta e = C_c \log \frac{\sigma'_f}{\sigma'_{CPT}}$ (Fig 25)

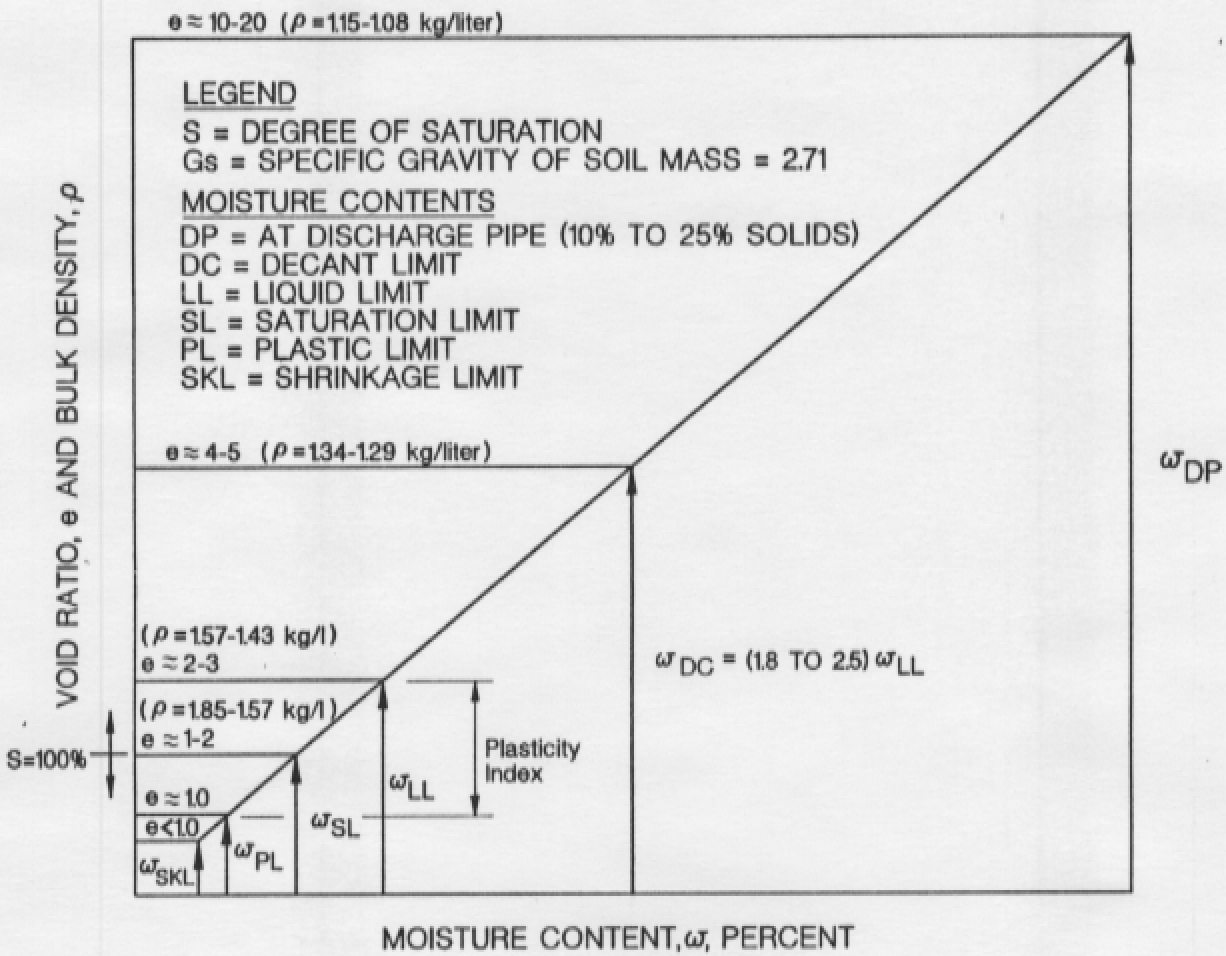


Figure 24. Moisture Content-Void Ratio Relationship for Craney Island Dredged Material

Compression Index

42. Figure 25 presents a summary of oedometer tests reported in the General Design Memorandums (U.S. Army 1949 and 1986). It can be seen that it is difficult to estimate a value of compression index (C_c) for the marine clay from this data. Ishibashi et al. (1992) suggested a value of C_c equal to 1.362 (Figure 25). Terzaghi and Peck (1967) presented the following empirical correlation for clay of medium to low sensitivity:

Conflicts w/ EM1110-1-1904, Pg 3-34. which states this is for organic clay w/sensitivity < 4.

$$C_c = 0.009*(LL - 10\%) \quad (10)$$

It gives $C_c = 0.01*(LL - 13\%)$ for clays.

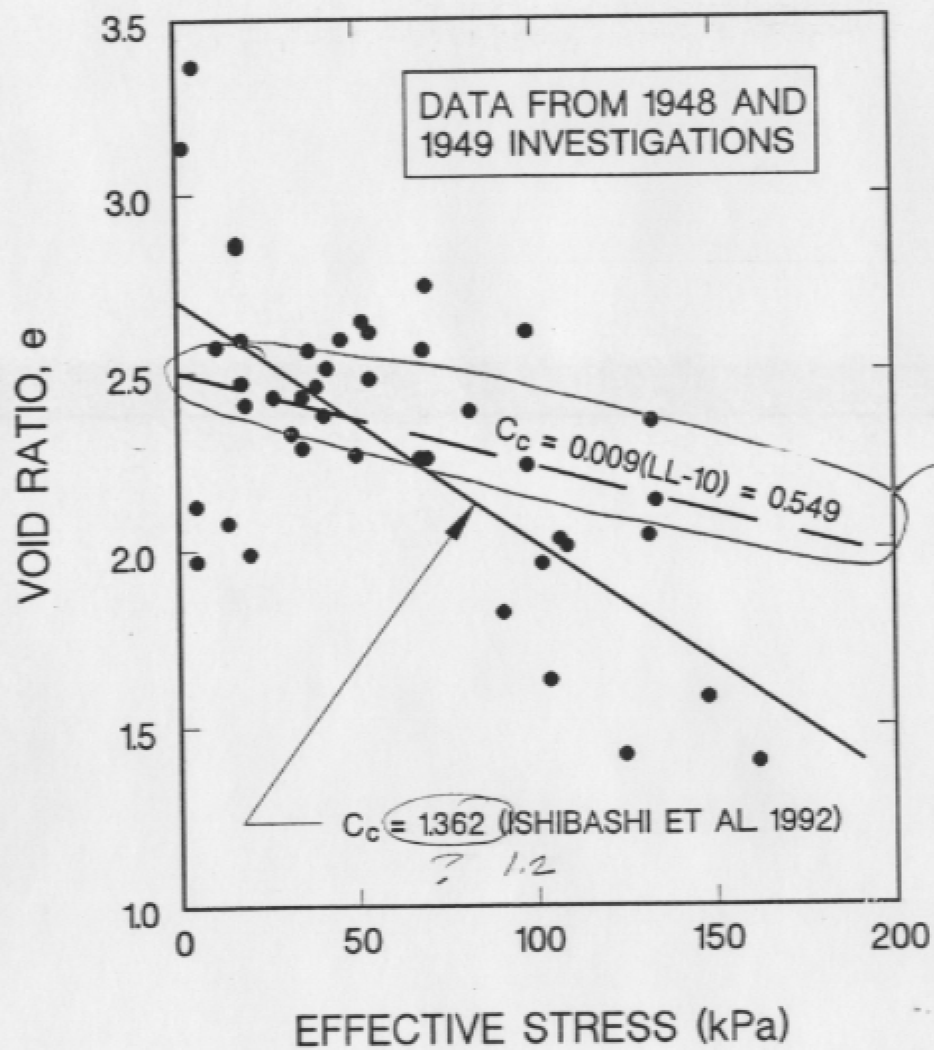
This equation and a liquid limit of 71 were used to estimate a value of C_c equal to 0.549. It can be seen from Figure 25 that the range in C_c is large. Both values of C_c were used in a subsequent section to estimate the consolidation settlement induced by installation of strip drains.

Coefficient of Consolidation

43. Strip drain spacing is governed by the vertical (C_v) and horizontal (C_h) coefficients of consolidation. It can be seen from Figure 7 that strip drains will penetrate the dredged fill and marine clay which have different hydraulic conductivities. These soil types are similar, but the void ratio of the dredged fill is larger than the marine clay. This results in a higher hydraulic conductivity and coefficient of consolidation for the dredged fill than the marine clay. The results of the subsurface investigation were used to estimate design values of C_v and C_h for the dredged fill and marine clay.

44. The results of hydraulic conductivity tests in piezometers installed in the perimeter dikes were used to estimate C_v and C_h. In these tests, water in the piezometers is either pumped down or raised by filling. After pumping or filling is completed, the time required for the water level to return to the original or equilibrium condition is measured. The flow around the piezometer tip is probably a combination of vertical and horizontal flow. However, for simplicity the flow was assumed to be horizontal and thus the hydraulic conductivity tests were assumed to be measuring the horizontal hydraulic conductivity.

45. The value of horizontal permeability is calculated from these hydraulic conductivity tests using the following equation (British Standards Institution 1981):



How was line fitted?

$$C_c = \frac{e_1 - e_2}{\log \frac{\sigma_2'}{\sigma_1'}}$$

Figure 25. Void Ratio-Effective Stress Relationship for Marine Clay (after Ishibashi et al. 1992)

Are these Cc independent of units?

Check Cc against ODU report

$$K_h = \frac{1.45 * A}{(t_2 - t_1)} * \log \frac{H_1}{H_2} \quad (11)$$

where K_h is horizontal hydraulic conductivity, t is time, A is the cross sectional area of the standpipe, and H is the variable head at times t_2 and t_1. The values of C_h were calculated for the dredged fill using the horizontal hydraulic conductivity from Equation (11), an initial void ratio, e_o, of 3 (Figure 23), a unit weight of water equal to 9.8 kN/m^3, a horizontal coefficient of compressibility, a_h, of 9.1E-03 (kPa)$^{-1}$, and the following equation:

?

$$C_h = \frac{K_h * (1 + e_o)}{a_h \gamma_w} \quad (12)$$

What kv values from these tests?

46. The value of a_h was obtained from oedometer and self weight consolidation test results (Cargill 1983) in the proper range of effective stress. The average horizontal hydraulic conductivity from three hydraulic conductivity tests in piezometers located in the dredged fill was estimated to be 2.4E-03 m/day. This hydraulic conductivity corresponds to an average value of C_h of 3.7E-02 m^2/day.

47. The values of C_h were calculated for the marine clay using the average horizontal hydraulic conductivity from field hydraulic conductivity tests in piezometers, an initial void ratio of 2 (Figure 23), and a coefficient of compressibility of 1.9E-02 (kPa)$^{-1}$. The average horizontal hydraulic conductivity from three hydraulic conductivity tests in the marine clay was estimated to be 7.2E-04 m/day, which corresponds to an average C_h of 1.2E-02 m^2/day. As expected, the marine clay exhibited a lower horizontal hydraulic conductivity and C_h because of the smaller initial void ratio.

why so low. On fig 2, you show e_{avg} = 2.5.

48. Installation of vertical strip drains, and thus disturbance, decreases the hydraulic conductivity of the soil, such that the ratio of the horizontal hydraulic conductivity to the vertical hydraulic conductivity ranges from 1.0 to 1.5 in marine clays (Mesri and Lo 1991). In undisturbed soil, this ratio can range from 3 to 10. Based on the data presented by Mesri and Lo (1991), a value of C_v was estimated by dividing C_h by an

average ratio of 1.25. Therefore, the values of C_v for the dredged fill and marine clay were calculated to be 3.0E-02 and 9.3E-03 m^2/day, respectively (Table 3).

Table 3. Estimated Values of C_v and C_h for the Dredged Fill and Marine Clay

Source of Data	C_v (m^2/day)	C_h (m^2/day)
Dredge Fill Data		
Field Piezometers (1991)	3.00E-02	3.70E-02
Cargill (1983)	8.80E-03	1.10E-02
Marine Clay Data		
Field Piezometers (1991)	9.30E-03	1.20E-02
Design Memorandums (1949 & 1986)	1.50E-03	1.90E-03
Empirical Correlations (U.S. Navy 1982)	7.90E-03	9.90E-03
Design Parameters	9.29E-03	1.16E-02

49. Oedometer test results from General Design Memorandums (U.S. Army 1949 and 1986) were also used to estimate C_v. Thirty-two time curves corresponding to the average effective stress in the marine clay (approximately 60 kPa) were used to estimate an average value of C_v for the test section area. The resulting average value of C_v is 1.5E-03 m^2/day. An average value of C_h was estimated to be 1.9E-03 m^2/day by multiplying C_v by 1.25.

50. It can be seen from Table 3 that the oedometer test results from the General Design Memorandums (U.S. Army 1949 and 1986) yielded values of C_v and C_h that are lower than the field hydraulic conductivity test values. The difference is attributed to sample disturbance, the lack of a representative sample, the accuracy of evaluating hydraulic conductivity without flow quantity or pore water pressure measurements in oedometer tests, and the combined vertical and horizontal flow that probably occurred around the piezometers during the field hydraulic conductivity tests.

51. Laboratory consolidation data on the dredged fill reported by Cargill (1983) were also used to obtain values of C_v and C_h equal to 8.8E-03 and 1.1E-02 m^2/day, respectively, for a void ratio of 3. Since these values of C_v and C_h were obtained from oedometer tests on the dredged fill at void ratios of approximately 3 (Cargill, 1983), these values should be similar to the field hydraulic conductivity tests. The discrepancy may be related to soil disturbance, differences in soil type, and the presence of thin drainage layers around the piezometers.

52. Unfortunately, the piezocone dissipation test results are not suitable to estimate C_v and C_h for the dredged fill and marine clay. Dissipation tests were performed at various depths during cone penetration testing at three locations in the test section. The pushing of the cone through the soil creates shear induced excess pore-water pressures, which had not completely dissipated when the test was stopped. Theories relating dissipation time to C_h and C_v generally require the time required for 50 percent dissipation or consolidation. In the field, pore-water pressure versus time was plotted on an arithmetic scale and dissipation seemed complete. However, the end of primary consolidation, and thus 50 percent consolidation, could not accurately be determined when the results were plotted using a semi-logarithmic scale.

53. Empirical correlations of C_v presented in the Navy Design Manual DM-7.1 (U.S. Navy 1982) were also used to estimate a value of C_v equal to 7.9E-03 m^2/day for the normally consolidated marine clay. This value of C_v corresponds to a value of C_h equal to 9.9E-03 m^2/day. Since the dredged fill is under going self-weight consolidation, values of C_v and C_h could not be estimated from this correlation. The values of C_v and C_h reported in this correlation correspond to effective stresses greater than those present in the dredged fill. Therefore, the dredged fill values of C_v and C_h are probably higher than those reported in the DM-7.1 correlation.

54. From Table 3 it can be seen that the values of C_v and C_h are uncertain. To facilitate the design of the test section it was decided to treat the dredged fill and marine clay as a single layer and use an average value of C_v and C_h. For design purposes, it was decided to use a weighted average value based on the thickness of the dredged fill and marine clay of C_v and C_h. The estimated average values of C_v and C_h are equal to 9.29E-03 and 1.16E-02 m^2/day, respectively, and were used to determine the preliminary spacing of the strip drains.

P 11

Strip Drain Design Parameters

55. The other major parameters required to develop an estimate of strip drain spacing are the well resistance and the extent of the smear zone. It can be seen from Figure 12 that the well resistance is governed by the ratio of K_h/K_w or K_h/q_w. Using field case histories, Lo (1991) showed that the effect of well resistance can be neglected if the parameter G is less than 0.2. Typical values of strip drain discharge capacity, q_w, range from 5.7 to 11.3 m^3/day (Koerner 1990). Since the consolidating clay is doubly drained, the maximum drainage length of the strip drain in the test section area is equal to 22 m. Using these parameters, an average value of q_w equal to 5.7 to 11.3 m^3/day, and the average horizontal hydraulic conductivity measured in the field piezometers, the value

ℓm

same as Hdr.

Editing to needed to correlate w/ Table 4.

See Table 4 Pg 28.
Need to mention $q_{w\,avg}$ = 8.5

of G ranges from 0.06 to 0.03. Therefore, well resistance may be neglected if the field discharge capacity of the strip drains is greater than 5.7 m^3/day.

56. The radial extent of the smear zone was studied using laboratory model tests by Onoue et al. (1991) and experience from pile driving and sand drain installations. This study revealed that the ratio of smear zone diameter to strip drain diameter, d_s/d_w, varies from 1.6 to 4.0. For design purposes the ratio of d_s/d_w was assumed to be 2. In addition, the horizontal hydraulic conductivity in the smear zone, K_s, was assumed to be one-half of the undisturbed hydraulic conductivity, K_h. This assumption is based on data presented by Onoue et al. (1991) that showed that the ratio of K_s/K_h ranged from 0.2 to 1.0 in the smear zone.

Design of Test Section Strip Drains

57. Using the design theory presented by Lo (1991) and the design parameters ~~previously~~ described (Table 4), a value of d_e equal to 2.3 m is required to obtain a degree of consolidation of 90 percent in the dredged fill and foundation clay within one year. The value of d_e is obtained by an iterative process in which values of d_e are selected until Equation (5) yields a degree of consolidation of 90 percent. The area influenced by each vertical strip drain is calculated using the following equation: See Pg B-2.

– From where? Lo?
Review Hausbo

$$t_{90} = \Pi\left(\frac{d_e}{2}\right)^2 \qquad (13)$$

58. Therefore, the area influenced by a vertical strip drain is the same for a square or triangular pattern. However, a triangular pattern provides better drainage for a specified area. The radius of influence does not reach the corners of a square area, and thus the square pattern will require a slightly longer time to consolidate than a triangular pattern. Since the area influenced by a square and triangular pattern is the same, it is recommended that a triangular pattern be used to facilitate drainage. A preliminary strip drain spacing for a triangular pattern was calculated to be 2.1 m by dividing d_e by 1.05.

59. The major design constraints for the test section were cost and the time required for 90 percent consolidation. Initially, it was decided that a consolidation time of 9 months was desired so that evaluation of the test section could be completed before specifications for a subsequent strip drain construction ~~of the second strip drain test section~~ contract would be required.

~~The second test section will be constructed during the 1993 summer parallel to the west perimeter dike. The main objective of the second test section is to investigate the effect of strip drains, and thus consolidation, on the settlement and stability of the west perimeter dike~~. To achieve 90 percent consolidation in nine months, a triangular pattern with a drain spacing of 1.8 m was recommended.

TABLE 4. Strip Drain Test Section Design Parameters

Parameter	Value
Degree of Consolidation	90%
Time	1 year
K_h	7.3E-04 m/day
C_v	9.3E-03 m2/day
C_h	1.1E-02 m2/day
H_{dr}	22 m
q_w	8.5 m3/day
l_m	22 m
d_s/d_w	2
K_s/K_h	0.5

Does not agree w/ Pg 24 - 7.2 1.2

Pg 27 Pg 27 Pg 24 Pg Pg Pg Pg

Not previously described. See Pg 26.

60. The lowest bid for the strip drain installation with a drain spacing of 1.8 m exceeded the project budget. As a result, the drain spacing was increased to 2.1 m to meet the project budget. However, twelve to thirteen months will be required to achieve 90 percent consolidation with a drain spacing of 2.1 m. Installation of the strip drains was completed in February, 1993, and thus 90 percent consolidation will be achieved in February or March, 1994.

I DONT THINK SO (HINDSIGHTS GREAT ... ISNT IT?)

Strip Drain Installation and Equipment

61. Vertical strip drains were installed in the test section using a novel piece of equipment. The equipment minimized disturbance to the sand blanket, confined dredged material, and the underlying marine clay during the installation operation. The equipment was developed by Geotechnics America, Inc. of Atlanta, Georgia (Figure 26). It can be seen that the 49 m high mandrel is stabilized using guy wires. The strip drain installation equipment had to be mounted on pontoons to reduce the maximum contact pressure to less than or equal to 10.4 kPa. This would enable the equipment to operate on the 15.2 to 30.5 cm thick desiccated crust in the mobility test section. The contractor mounted the installation equipment on two 2.1 m wide and 10.7 m long pontoons (Figure 27). The entire equipment weighs approximately 45,000 kg. Since the area of the pontoons is 45.6 m^2, the ground pressure exerted by this equipment is only 9.7 kPa.

Report in SI units. If discussing weight (force) then need Newtons. If discussing mass, then need to change wording.

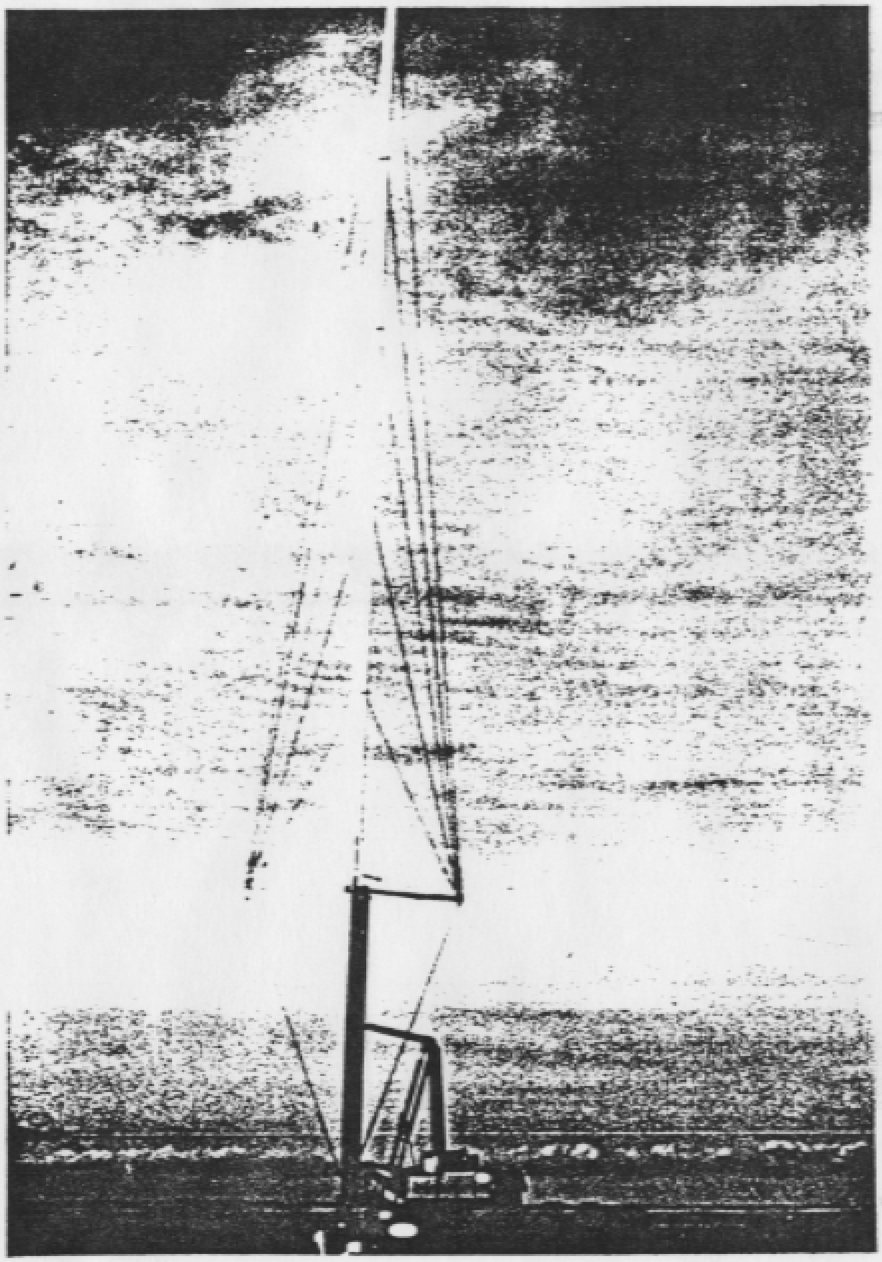


Figure 26. Overview of Strip Drain Equipment

Report in SI format
Express values in m. or mm.

62. The drains were advanced using a mandrel sleeve that was pushed through the sand blanket, dredged material, marine clay, and into the dense sand. The mandrel protects the drain material from tears, cuts, and abrasions during installation. The cross-sectional area of the mandrel was restricted to 64.5 cm^2 to reduce soil disturbance during installation. The mandrel is retracted after each drain is installed at the required depth. A flat anchor plate is placed at the bottom of the mandrel to prevent soil from entering the bottom of the mandrel, to minimize tearing of the geotextile, and to anchor the drain material at the required depth when the mandrel is retracted.

63. The depth to the dense sand in the test section is approximately 40 to 49 m. The vertical strip drains had to be anchored in the dense sand underlying the marine clay to ensure that the drains would be doubly drained. To achieve this objective, each drain was pushed until a pressure of 69,000 kPa was applied to the mandrel. This was the maximum pressure that could be applied without lifting the equipment from the ground surface.

64. The static method of installation with a constant rate of advancement was used for advancing the drains to reduce soil disturbance. An advancement rate less than 9.0 m per minute with the full static force was used. Approximately 10 cm to 20 cm of strip drain material was left protruding above the sand blanket in the main test section. In the mobility test section, a much longer length of strip drain was left protruding above the desiccated crust so that the vertical strip drain could be connected to the nearest horizontal strip drain. Horizontal strip drains were utilized in the mobility test section to evaluate their ability to convey water to the perimeter trenches. Horizontal strip drains were not used in the main area because the sand blanket provided drainage to the perimeter trenches. In addition, the sand blanket and horizontal strip drains will provide an intermediate drainage layer as additional dredged material is placed in the area. The successful use of horizontal strip drains would preclude the cost and installation of a sand blanket throughout the remainder of the management area.

Same comment above

65. Strip drain installation in the test section began on 21 December 1992 and was completed on 19 February 1993. The total number of drains installed in the main and mobility test sections is 5,557. Approximately 193,824 lineal meters of vertical strip drain were installed in the main section while 40,755 lineal meters of vertical strip drain were installed in the mobility section. In the mobility section 2,181 lineal meters of horizontal strip drain were installed. The successful bid for the strip drain installation utilized the unit costs shown in Table 5.

Conflicts w/ Pg 15 see pg 34.

Table 5. Strip Drain Costs for Craney Island Test Section

Item	Unit Price ($/m)	Quantity Installed (m)	Item Cost ($)
Vertical Strip Drains			
- Main Section	1.98	193,824	383,772
- Mobility Section	4.26	40,755	173,616
Horizontal Strip Drains			
- Mobility Section	49.18	2,181	107,262
		Total Project Cost:	$664,650

66. It can be seen that the unit pricing of the vertical strip drains in the mobility section is about two times higher than the main section. This was attributed to the uncertainty of operating the installation equipment on the desiccated crust. The low ground pressure of the installation equipment enabled the contractor to operate efficiently on the desiccated crust. As a result, it was concluded that a sand blanket is not required throughout the disposal area to permit drain installation. This achieved one of the major objectives of the test section, which was to determine if strip drains could be installed without the use of a sand blanket. Based on the successful installation of strip drains in the mobility section, the unit price for installing vertical strip drains in a subsequent contract ~~the second test section~~ is only $1.98 per lineal meter (Joiner 1993). This is the same cost as the strip drains in the main test section. Therefore, the absence of a sand blanket will not affect the cost of future strip drain installation.

Strip Drain Specifications

67. The vertical strip drains consist of a band-shaped plastic core enclosed in a suitable filter material. The polypropylene drainage core is wrapped in a filter made of a non-woven fabric of continuous filaments of 100% polypropylene. Strip drains with nipples or other individual protruding objects used to create a drainage channel were not specified for this project. It was anticipated that the lateral pressures in the marine clay are large and could cause the filter fabric to be punctured by protruding objects on the drainage core. The initial strip drain specifications also required the following physical characteristics (Units as specified):

DRAIN:		
Weight	126 grams/meter (0.085 lbs/ft)	
Width	93 mm (3.7 inches)	
Thickness	4.1 mm (0.16 inch)	
Roll Length	305 m (1000 ft)	
Discharge Capacity	6.4×10^{-3} m^3/min (1.6 gpm)	ASTM D4716 ($345 kN/m^2$, 50psi)
Discharge Capacity	6.4×10^{-3} m^3/min (1.6 gpm)	ASTM D4716 (25% compression)
CORE:		
Material	Polypropylene	
Drainage Channels	54	
Grab Tensile Strength	21,390 kN/m^2 (3100 psi)	ASTM D1621/D638
Flex Modulus	1.1 kN/m^2 (0.16 psi)	
Density	0.90 g/cm^3 (56.2 pcf)	ASTM D792
One Side Wetted Perimeter	19.2 cm (7.56 in)	
FILTER FABRIC:		
Grab Tensile Strength	0.89 kN (200 lbs)	ASTM D4632
Grab Elongation at Break	60%	ASTM D4632
Modulus at 10% elongation	5.3 kN (1,200 lbs)	ASTM D1682/D4632
Trapezoidal Tear	0.33 kN (75 lbs)	ASTM D4533
Puncture Strength	0.31 kN (70 lbs)	ASTM D4833
Mullen Burst Strength	1373 kN/m^2 (210 psi)	ASTM D3786
Specific Gravity	0.95	
Permittivity	3238 lpm/m^2 (230 gpm/ft^2)	ASTM D4491
Permeability (K)	0.01 cm/min (0.0039 in/min)	ASTM D4491
Ultra Violet Resistance	70% at 500 Hours	ASTM D4355
Apparent Opening Size (AOS)	140	ASTM D4751

68. The applicable American Society for Testing and Materials (ASTM) standard test designations (Annual 1992) are presented in the specification. These specifications were developed to reduce the potential for the filter fabric to be squeezed into the channels of the drainage core. If the filter fabric was forced into the drainage channels,, the discharge capacity of the drain would be significantly reduced. To reduce this possibility, a large number of drainage channels was specified (greater than or equal to 54) to reduce the area that the filter fabric had to span. A typical strip drain, for example, Amerdrain 407, has less than or equal to 40 drainage channels. In addition, a heavier filter fabric (186 g/m) was specified to resist the large lateral pressures in the marine clay. A heavier filter fabric also provides better flow characteristics. The thicker filter fabric reduces the amount of soil particles entering the drainage channels, which helps maintain the discharge capacity of the drain. The Amerdrain 410, manufactured by American Wick Drain Company, was proposed by the contractor to satisfy the specifications shown above.

69. In an effort to reduce the cost of the test section after the first bid, NAO inserted the following strip drain specification:

DRAIN:

Weight	0.80-1.30 N/Meter	
Width	90-105 mm	
Thickness	3-4 mm	
Discharge Capacity	60 mil/sec	ASTM D4716

GEOTEXTILE:

Grab Tensile Strength	720 N	ASTM D4632
Puncture Strength	270 N	ASTM D4833
Water Permeability (K)	> 0.01 cm/sec	ASTM D4491

70. The main difference in the two specifications for vertical strip drains is that the second specification requires a plastic drainage core with greater than 38 channels and a filter fabric weight of only 124 g/m. This resulted in the contractor installing the Amerdrain 407 manufactured by American Wick Drain Company.

71. The horizontal strip drains were also prefabricated with a polypropylene drainage core wrapped in a filter. The filter fabric is made of non-woven fabric of continuous filaments of 100 percent polypropylene. The contractor used a 10.2 cm wide Akwadadrain, manufactured by American Wick Drain Company, to satisfy the following specification for horizontal strip drains.

Use SI Units.

DRAIN:

Weight	2400 g/m^2 (7.9 oz/ft^2)	
Width	305 mm (12 inches)	
Thickness	25 mm (1 inch)	
Compressive Strength	430 kN/m^2 (9000 psf)	ASTM D695/D1621
Shear Strength	430 kN/m (9000 lbs/ft)	ASTM D1621
Peel Strength	14.3 kN/m (35 lbs/ft)	ASTM D1876
Fungus Resistance	No Growth	ASTM G21
In Plane Discharge Capacity	0.0013 m^3/sec (20 gpm/ft width)	ASTM D4716 (Gradient = 0.1 at 10 psi)

CORE:

Tensile Strength	0.6 kN (135 lbs)	ASTM D638
Fungus Resistance	No Growth	ASTM G21

FABRIC:

Weight	1190 g/m^2 (3.9 oz/yd^2)	ASTM D3776
Grab Tensile Strength	0.5 kN (110 lbs)	ASTM D4632
Elongation at Break	40%	ASTM D4632
Modulus at 10% elongation	5.3 kN (1,200 lbs)	ASTM D1682/D4632
Trapezoidal Tear	0.22 kN (45 lbs)	ASTM D4533
Puncture Strength	0.33 kN (75 lbs)	ASTM D4833
Mullen Burst Strength	1518 kN/m^2 (220 psi)	ASTM D3786
Flow	1410 lpm/m^2 (100 gpm/ft^2)	ASTM D4491
Permeability (K)	0.15 cm/sec (0.059 in/sec)	ASTM D4491
Ultra Violet Resistance	80% at 500 Hours	ASTM D4355
Equivalent Opening Size	70	ASTM D4491
Fungus Resistance	No Growth	ASTM G21

PART IV: TEST SECTION PERFORMANCE

Use SI units - millimeters.

72. Immediately following installation of the vertical strip drains, water could be seen rising in the drainage core and around the strip drain. The water rising around the strip drain was due to the void left by the mandrel after retraction. Figure 28 is a photograph of a typical strip drain within 10 to 20 minutes after installation. It can be seen that water has risen several centimeters above the ground surface inside the drain. A significant amount of water can be seen rising around the drain. This void around the drain will close shortly due to the lateral earth pressures in the dredged fill and marine clay. Subsequently, flow will only occur in the drain

Estimated Magnitude of Consolidation Settlement

73. Several techniques were used to estimate the magnitude of consolidation settlement that will occur in the test section. First, consolidation settlement was calculated for the marine clay by assuming instantaneous placement of the dredged fill at Craney Island. This case is unrealistic since it assumes instantaneous placement of the surcharge and does not account for self-weight consolidation of the dredged fill. However, it provides an upper bound estimate of the expected settlement of the marine clay. The change in effective stress and Figure 25 were used to estimate the change in void ratio, and thus consolidation settlement. For C_c equal to 0.549 the estimated settlement is 2.0 m, while for C_c equal to 1.362 the expected settlement is 5.1 m.

Need to explain how develop final effective stress. Is it same as described in next pg, then should be described here first and refer to in next para.

74. Consolidation settlement estimates were also made using the undrained shear strength data obtained from cone penetration tests conducted prior to strip drain installation. The variation in S_u with depth (Figure 20) was determined from cone penetration tests using a value of N_k equal to 12. The current effective stress profile was estimated by dividing the values of S_u by an average value of S_u/σ_p' equal to 0.26. The difference between the current effective overburden stress distribution and the final effective stress distribution equals the increase in effective stress at 100 percent consolidation. Using values of C_c equal to 0.549 and 1.362, a range of consolidation settlement of 0.7 m to 1.9 m was estimated. Since settlement of the test section already exceeds 1.2 m this technique appears to underestimate the consolidation settlement. This is probably caused by the difficulties in estimating the current effective overburden profile, that is, the current pore-water pressure profile.

75. Consolidation settlement was also estimated using the change in void ratio due to 100 percent consolidation. The current void ratio distribution (Figure 23) was determined from samples obtained from the boring at the center of the test section. Final

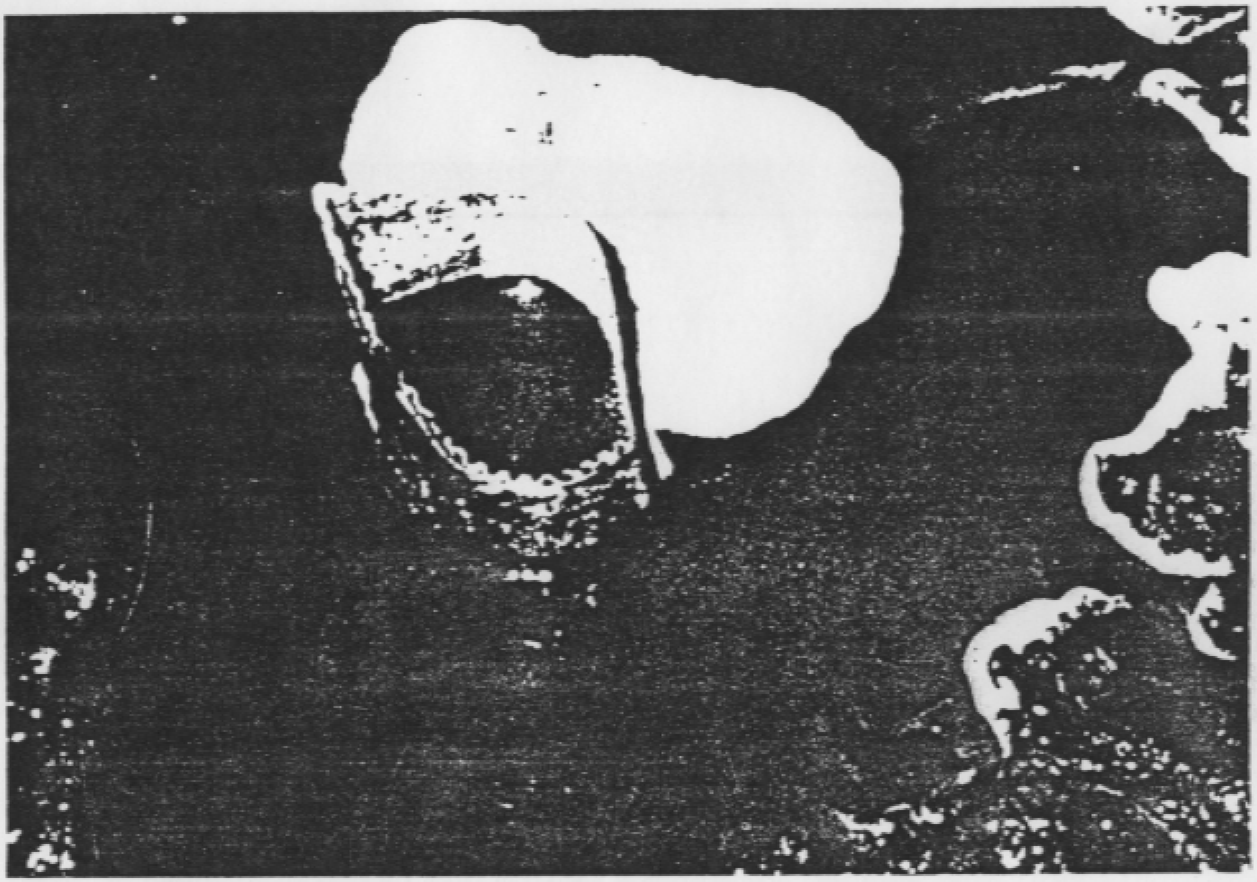


Figure 28. Water Rising in Strip Drain Immediately After Installation

CONFLICT. See Pg 20.

was used to estimate initial Su profile in Fig 20. See comment Fig 23.

void ratios, corresponding to 100 percent consolidation, were estimated using the final effective stress and the void ratio-effective stress relationship in Figure 25. The final effective overburden stress was estimated using a unit weight of $\boxed{14.6\ kN/m^3}$ for the dredged fill and marine clay and a dredged fill surface at El. +7.3 m MLW. Consolidation settlements of 1.9 m to 5.2 m were estimated for the values of C_c equal to 0.549 and 1.362, respectively.

76. Consolidation settlements estimated using the previously described techniques are summarized in Table 6. In summary, it is anticipated that the test section will settle between 1.9 m and 2.4 m before 100 percent consolidation is achieved.

How do you draw this conclusion; specifically, explain where upper range comes from

Table 6. Estimates of Consolidation Settlement for Test Section

Method	Cc = 0.549 Settlement (m)	Cc = 1.362 Settlement (m)
Instantaneous Placement	2.0	5.1
S_u/σ_p' = 0.26 Analysis	0.7	1.9
Change in Void Ratio	1.9	5.2

Conflict: See Pgs 15 & 29.

Measured Consolidation Settlements

77. Settlement plate readings for the main section and mobility section are presented in Figures 29 and 30, respectively. Installation of the vertical strip drains in the test section was completed on 19 February 1993. As of 6 July 1993 the maximum consolidation settlement in the mobility ~~test section~~ is approximately 0.7 ~~1.2~~ m. As a result, the measured settlements are still less than the predicted range of 1.9 m to 2.4 m. It is anticipated that consolidation will continue until February or March, 1994, and thus it appears reasonable to assume that the settlements will exceed 1.9 m.

or address effect of. Need to compare to mobility section. Estimated settlement did not take into consideration sand blanket. Also since no blanket will be used in future, the mobility section

78. It should be noted that strip drains were installed in the northern part of the test area first. As a result, the settlement plates in the northern portion of the main test section (SP-1, ~~SP-5~~, & SP-7) show a faster response than the other settlement plates. For example, settlement plates SP-1 and SP-7 show a significant decrease in elevation after only 20 to 25 days. Conversely, settlement plates SP-3 and SP-9 did not show a significant decrease in elevation until 40 to 50 days after strip drain installation commenced.

Located in Center.

79. The mobility section was developed to demonstrate that a sand blanket was not required to support the strip drain equipment. Since the equipment exerted a ground pressure of only 9.7 kPa, a sand blanket is not required for future strip drain installations.

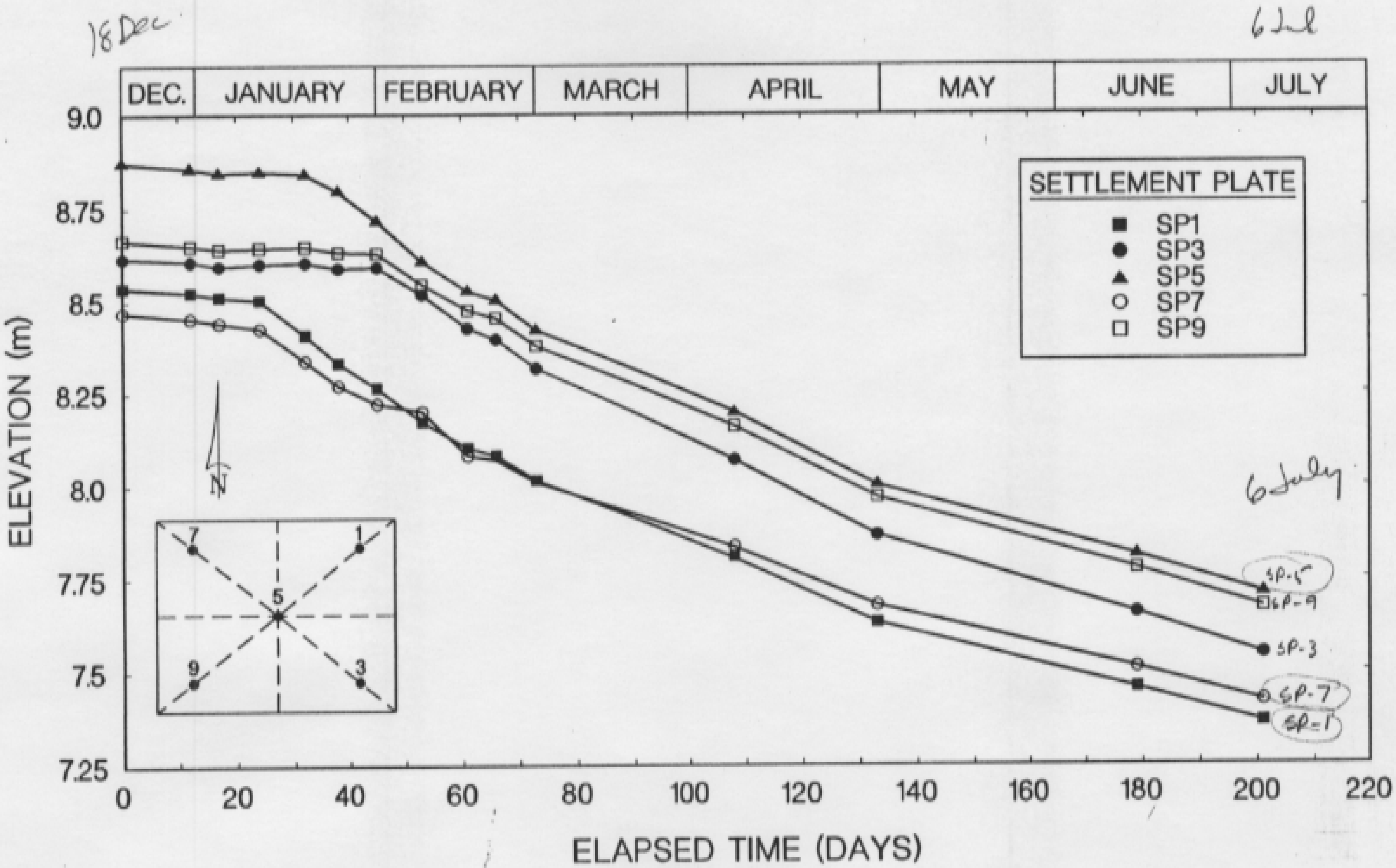


Figure 29. Settlement Plate Measurements in Main Section

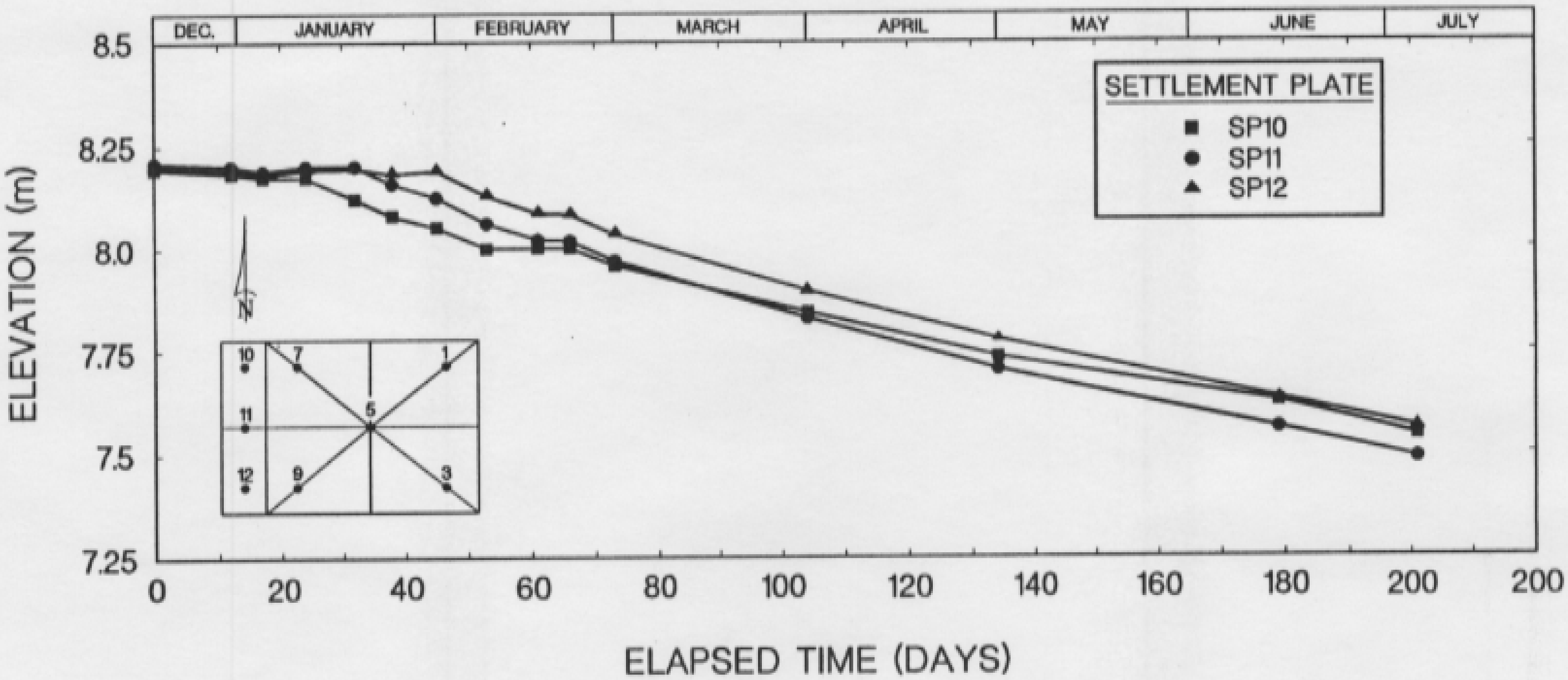


Figure 30. Settlement Plate Measurements in Mobility Section

A comparison of Figures 29 and 30 provides an insight into the effect of the sand blanket on consolidation of the dredged fill and marine clay. It can be seen that settlement plate SP-10 is located at the northern end of the adjacent mobility section and can be compared with settlement plates SP-1 and SP-7 at the northern end of the main section. Settlement plates SP-1 and SP-7 have settled 1.1 m to 1.2 m while settlement plate SP-10 has settled only 0.6 m. Therefore, it may be concluded that the additional surcharge provided by the sand blanket results in a significant increase in consolidation settlement. It is anticipated that the additional consolidation occurred in the dredged fill because of the limited extent of the sand blanket. In summary, the storage capacity lost by the installation of a sand blanket can probably be recouped by the subsequent consolidation of the underlying dredged fill. However, the cost of the blanket will probably preclude ~~to~~ the use of a sand blanket throughout the remainder of the strip drain test sections.

Include a predicted curve or band curved based on estimated ΔH from para. 76.

80. Figures 31 and 32 present the settlement plate data from the main and mobility section, respectively, using a semi-logarithmic scale. It can be seen that none of the settlement plates indicate that primary consolidation has been completed. This is in good agreement with the prediction that approximately twelve to thirteen months will be required for 90 percent consolidation. Figures 31 and 32 also illustrate the difference in settlement between the main section (1.1 m to 1.2 m) and the mobility section (0.6 m to 0.8 m).

See comment Pg 34. Estimated settlement did not address effect of blanket. Should compare settlement to SP-10. Also put in SI units.

Also define start date. Is it at 18 Dec or mid point of construction etc.?

Time Rate of Settlement

81. Figure 33 presents the measured and estimated time rate of consolidation settlement for the main section. The estimated curves were obtained using the consolidation properties in Table 4. It can be seen that there is excellent agreement between the measured and estimated time-rate of consolidation relationships. It also appears that twelve to thirteen months will be required to achieve 90 percent consolidation.

This is misleading because of the obvious effect sand blanket has. Plot SP-10 in Fig 33 and draw conclusion on both.

Excess Pore-Water Pressures

82. Figure 34 presents typical piezometric readings for the piezometers installed in the test section. Figure 34 presents the measurements from piezometer cluster located at the center of the main section (P-5). It can be seen that the piezometer data does not correspond to the consolidation settlements that are being measured. This trend has been noted by other researchers including (Hansbo et al. 1982). Hansbo et al. (1982) also showed that an increase in undrained shear strength was observed in several case histories with a negligible change in excess pore-water pressure. Based on these results, it is

ALSO SHEAR STRENGTH (CPT) DATA DOES NOT CORRESPOND

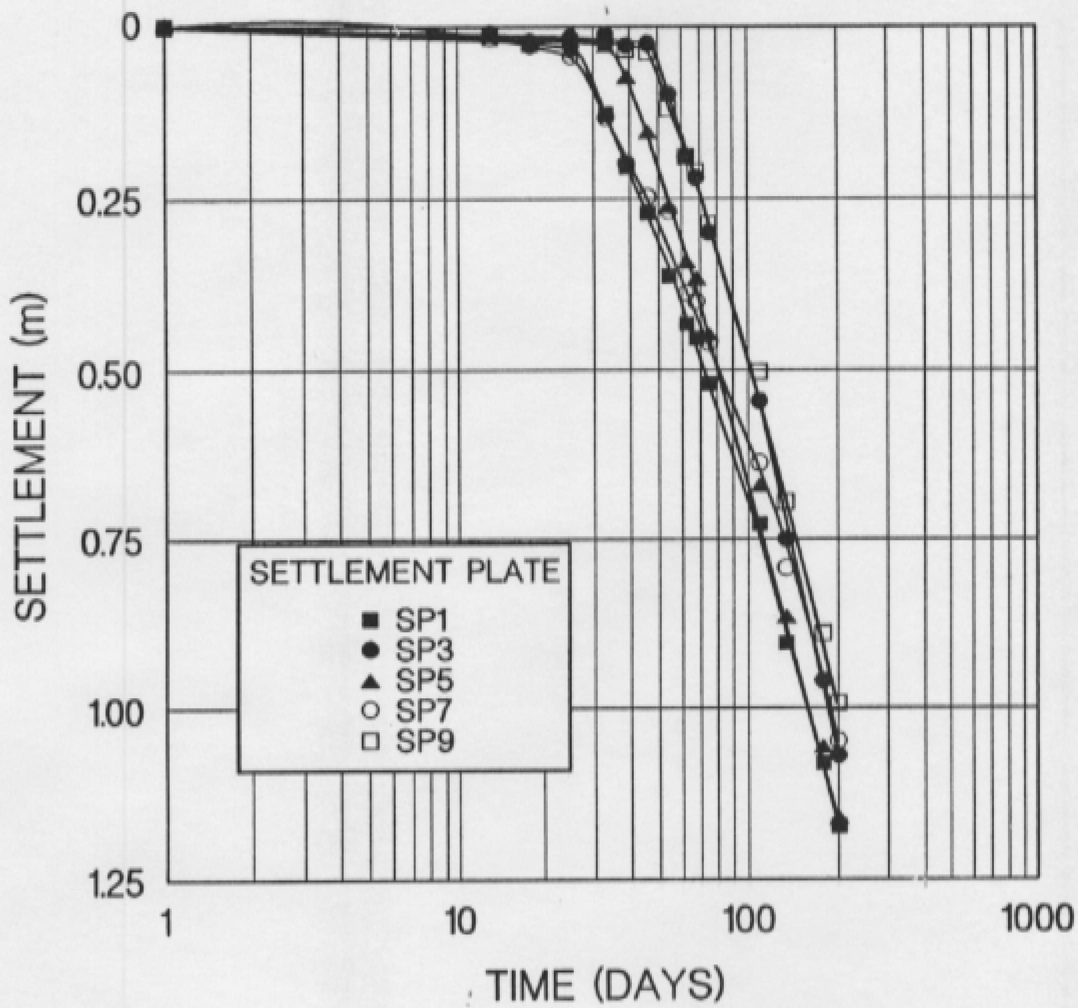


Figure 31. Semi-Logarithmic Presentation of Settlement Plate Measurements in Main Section

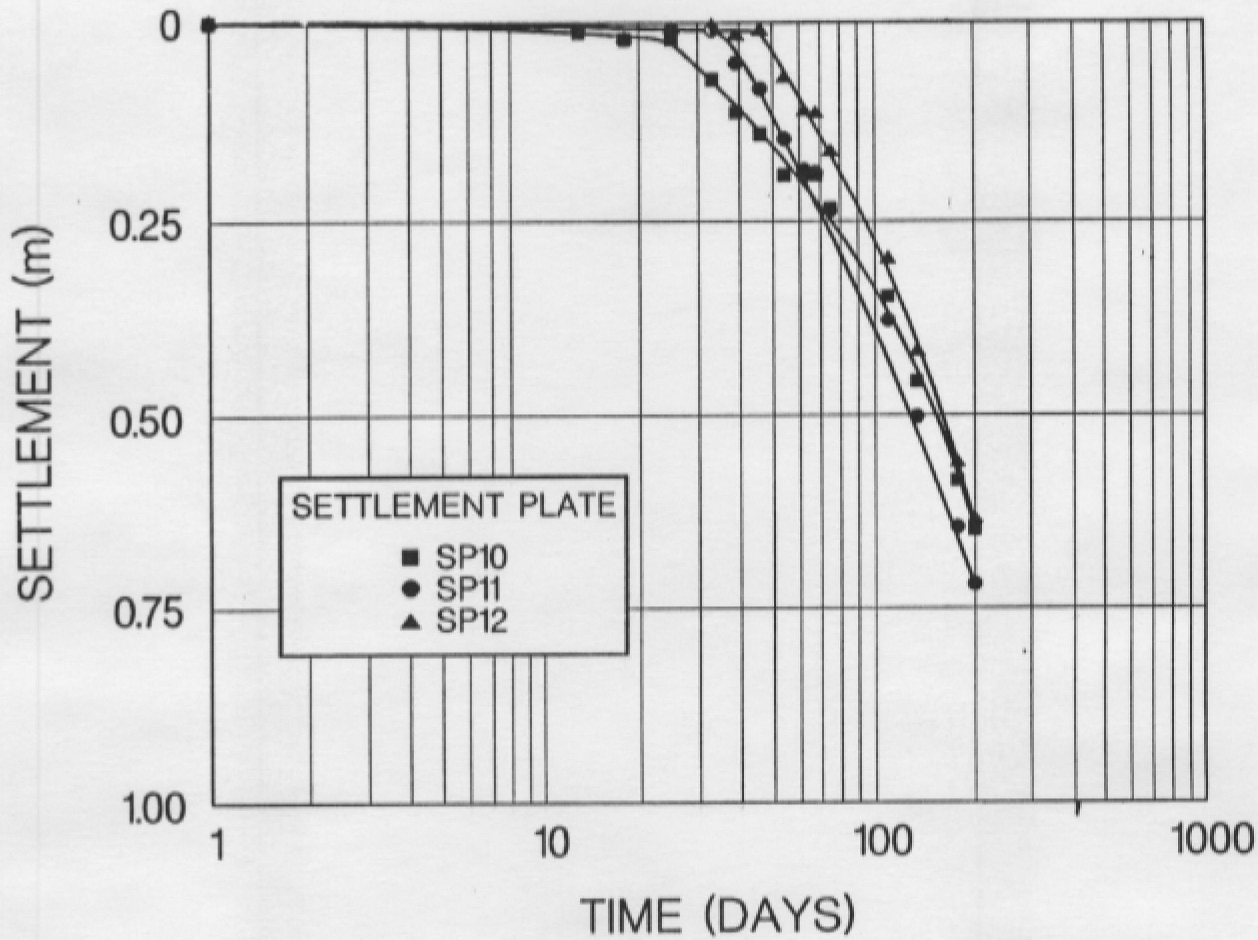


Figure 32. Semi-Logarithmic Presentation of Settlement Plate Measurements in Mobility Section

Explain
Start date or end date
Different from
Other graph.

Why units in english units?

Estimated settlement did not address surcharge. Should compare against SP-10. See comments Pg 34 & 35.

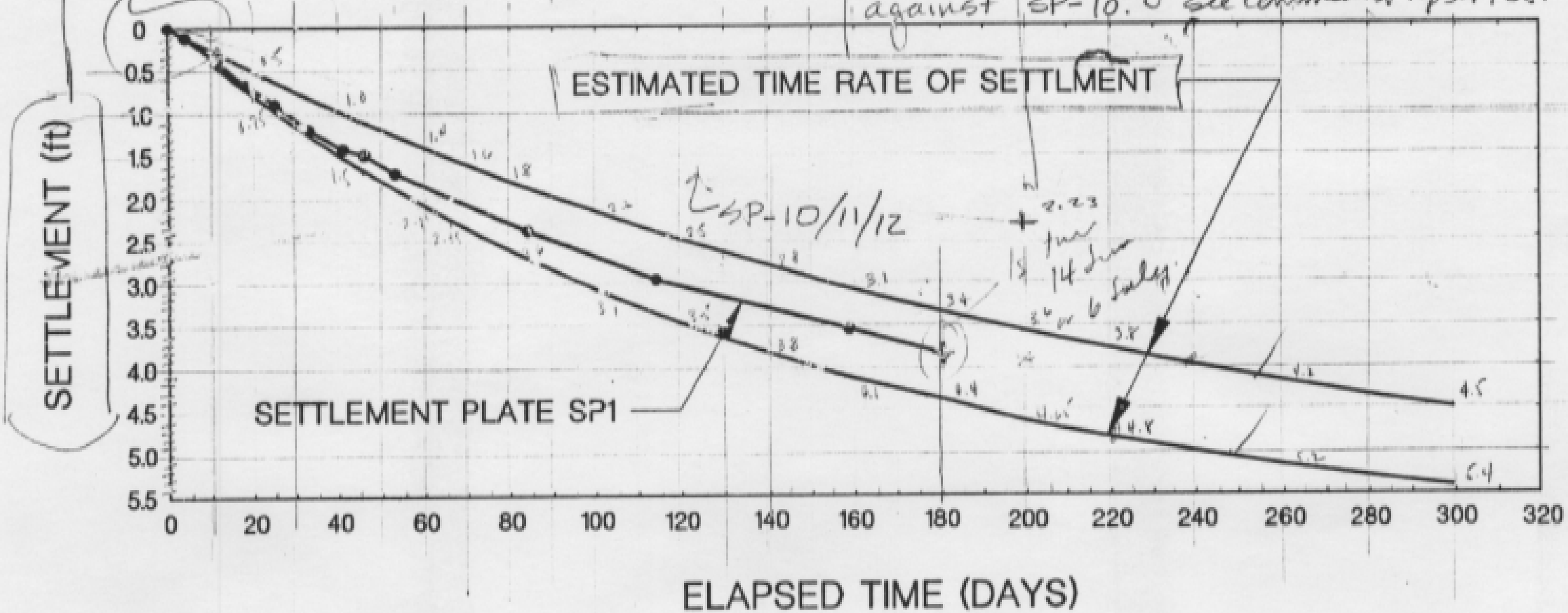


Figure 33. Measured and Estimated Time Rate of Consolidation Settlement for Main Test Section

Why in english units?

Craney Island Test Section - Piezometer Cluster P-5

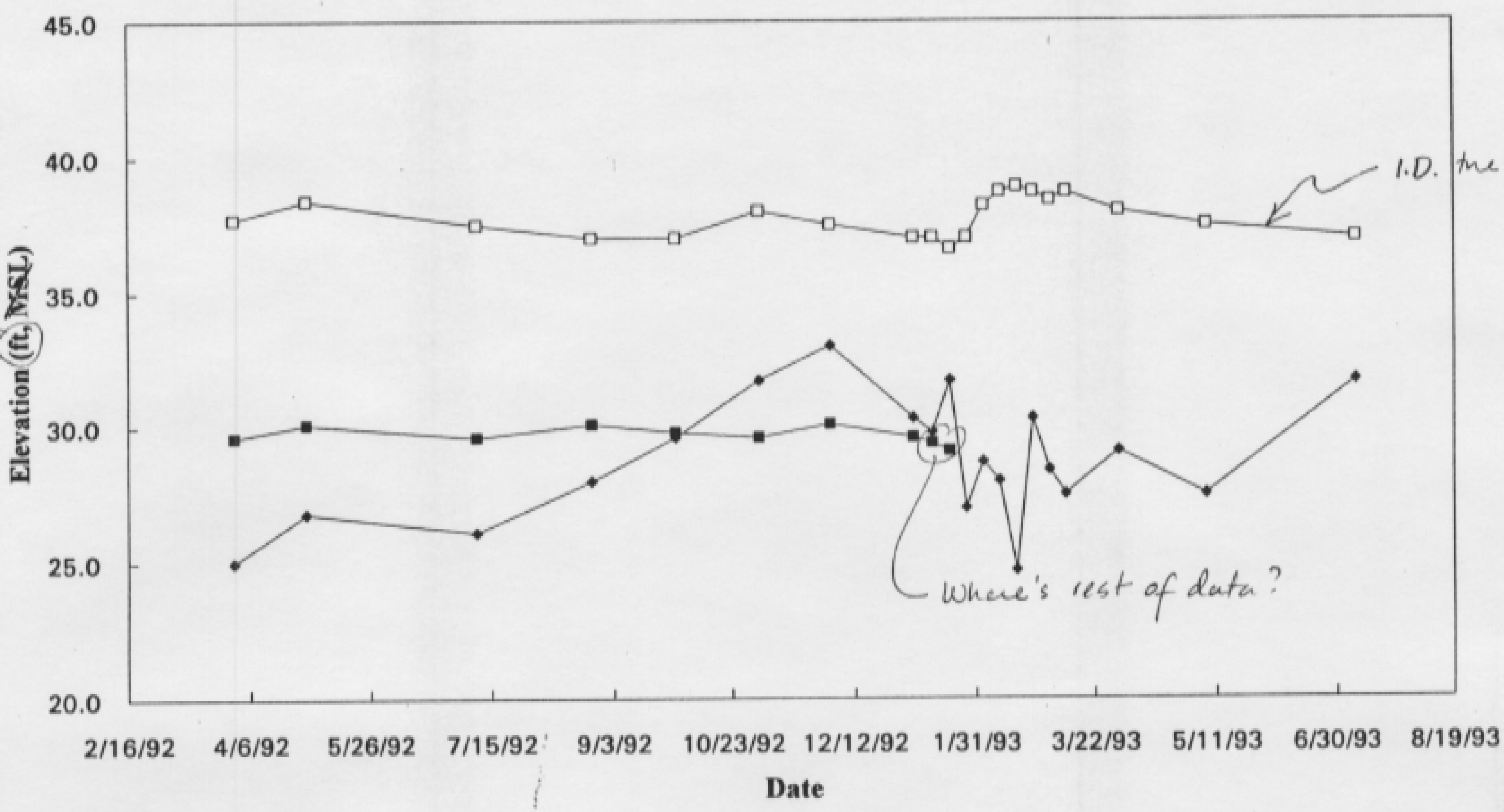


Figure 34. Typical Piezometer Measurements in Test Section

and settlement pts w/ depth

recommended that subsequent strip drain test sections should rely more on settlement plate measurements than on piezometers to evaluate the effectiveness of strip drains.

Undrained Shear Strength

83. The undrained shear strength profile at 100 percent consolidation was estimated using an undrained strength ratio of 0.25 to 0.27. The current and estimated final values of S_u are presented in Figure 20. It can be seen that a substantial increase (85 to 90 percent) in S_u is predicted for the marine clay.

POST CPT DOES NOT SUPPORT THIS

84. A smaller increase is estimated for the dredged fill because this soil is not significantly under-consolidated. The presence of sand and silt seams in the dredged fill has allowed the excess pore-water pressures induced by self-weight consolidation to dissipate. As a result, there probably will not be a large increase in S_u in the dredged fill after strip drain installation. Cone penetration and field vane shear tests will be conducted after consolidation has been completed to determine the increase in S_u in the dredged fill and marine clay.

General we aim for within 1.5 m.

Post Consolidation Subsurface Investigation

85. After consolidation has been completed in February or March, 1994, cone and piezocone penetration tests will be conducted within 6 m of the previous cone penetration test locations. In addition, a boring will be drilled within 6 m of the previous borehole to measure the new water content and profiles. The change in water content and penetration resistance will be related to the increase in S_u and the magnitude of settlement. Quantifying the magnitude of settlement and the increase in S_u will aid in determining whether installing strip drains in the three compartments of Craney Island is economically feasible.

PART V: SUMMARY AND CONCLUSIONS

A supplemental study by the PI will investigate the use of strip drains beneath exterior perimeter levees to improve stability conditions.

86. A 183 m by 122 m vertical strip drain test section was completed in February, 1993 in the north compartment of the Craney Island Dredged Material Management Area. The test section was constructed to evaluate the effectiveness of prefabricated strip drains in consolidating the dredged fill and underlying foundation clay, and thus increasing the storage capacity of the facility. The feasibility of installing strip drains was questionable since drains had never been installed in an active dredged material management area, a drain length of 49 m was close to the longest drain ever installed, and the installation equipment had to operate directly on the surface of the dredged material. Consolidation of the dredged fill and foundation clay will also cause a large increase in soil shear strength. ~~The strength gain will allow perimeter dikes to be constructed to a higher elevation without setbacks or stability berms.~~ It is anticipated that the strip drains will continue to function as additional dredged material is placed in the management area, and thus increase storage capacity in the future.

I don't think so

87. The subsurface investigation conducted prior to strip drain installation revealed that large excess pore-water pressures exist in the foundation clay. The excess pore-water pressures in the dredged fill are significantly smaller than the marine clay. The sand and silt seams identified from cone penetration test results probably dissipate the excess pore-water pressures induced by the self-weight consolidation of the dredged fill. As a result, the large consolidation settlements that are occurring at the test section are primarily attributed to the marine clay.

88. In the main test section a sand working pad was constructed while no sand pad was constructed in the mobility area. The main objective of the mobility section was to determine if the installation equipment could operate directly on the dredged material. The pontoon mounted equipment was successful in operating in the mobility section. This has significantly reduced the cost of vertical strip drains in the second test section. Comparison of the settlements in the mobility and main section revealed that the sand blanket caused an increase in settlement by surcharging the dredged material. Therefore, surcharging confined dredged material can lead to substantial consolidation and increase in storage capacity.

Is the actual settl't (i.e. increased storage capacity) greater than the thickness of sand working mat? If not, this conclusion is not valid

89. Settlement plates installed in the test section have settled approximately 1.2 m since February 1993. It is anticipated that 90 percent consolidation will be completed in February or March, 1994. The final consolidation settlement of the test section is estimated to be between 1.9 m and 2.4 m. So far, the predicted time rate of settlement is

in good agreement with field measurements. Cone and piezocone penetration tests will be conducted after consolidation is completed to quantify the increase in undrained shear strength.

90. In summary, the Craney Island test section showed that strip drains are an effective technique for increasing the storage capacity, and the service life, of confined dredged material management area. This technique appears to be applicable to many management areas around the world.

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